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UNITED STATES AIR FORCE SOIL STABILIZATION INDEX SYSTEM - A VALIDATION

Wayne A. Dunlap, et al

Texas A and M University

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Air Force Weapons Laboratory

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This technical report has been reviewed and is approved for publication.

Larry A. Dillon

LARRY A. DILLON
Major, USAF
Project Officer

FOR THE COMMANDER

Frederick H. Peterson

FREDERICK H. PETERSON
Chief, Aerospace Facilities Branch

William B. Liddicoet

WILLIAM B. LIDDICOET
Colonel, USAF
Chief, Civil Engineering Research Division

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This report covers the validation of a soil stabilization index system which was developed earlier. The index system was originated to aid military engineers in selecting the appropriate type and amount of soil stabilizer to use in pavement construction. A comprehensive review of literature in the soil stabilization field was used to initially develop the index system. Laboratory tests and discussions with experts in soil stabilization were used in the validation phase reported herein. Based on these tests and discussions, several changes have been made to the initial index system, although the original concept (OVER)

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has not been altered. The index system is entered with easily determined soil properties and flow charts are followed to arrive at the most suitable stabilizer. Subsystems containing appropriate tests are used to determine specific amounts of stabilizers. Use factors, construction factors and environmental factors are also considered in the decision-making process. Recommendations are included for additional verification studies of the index system.

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SECTION I

INTRODUCTION

1. Background

The U.S. Air Force owns and maintains about one half billion square yards of paved surface. New construction and strengthening of existing pavements for heavier loads are continuing processes. Any means of reducing initial or maintenance costs by improving the quality of the materials available for construction is a worthy endeavor even if the saving per square yard of surfacing is small. Also, combat requirements often demand rapid construction, and many times it is necessary to construct with native materials which under less stringent time requirements would be considered unsuitable.

Improving the engineering properties of earth materials, either by mechanical means, e.g., compaction, or by the addition of chemicals, is generally termed soil stabilization. It is a process which is accomplished -- to greater or lesser degree -- in the construction of all roads and runways. As a result, there is a wealth of experience on stabilization. Much of this experience has been translated to print in the area of mechanical stabilization: however, knowledge in the newer area of chemical stabilization is less well distributed.

As a result, engineers -- particularly military engineers faced with situations requiring rapid construction -- often are forced to make decisions about stabilization with meager information on which to base their decisions.

In an effort to upgrade the available techniques and information, the Air Force Weapons Laboratory (AFWL) embarked on a program of research in soil stabilization which included: applicability of resins for stabilizing soils (reference 1), determination of physico-chemical properties of soils influ-

encing stabilization (reference 2), in situ deep-layer stabilization of soils (reference 3), development of a soil stabilization index system (reference 4), and means of efficiently mixing soils and stabilizers.

2. Development of Soil Stabilization Index System

The Soil Stabilization Index System (SSIS) was developed at Texas A&M University to satisfy Air Force requirements for a systematic means of determining a soil's suitability for stabilization and the most appropriate type and amount of stabilizer for use with the soil. (Only lime, portland cement and bituminous stabilizers were considered.) Furthermore, SSIS was to be arranged in such a form and with sufficient background information that it could be effectively used even by engineers not specifically trained in stabilization techniques. Insofar as possible, SSIS was also to consider factors influencing soil stabilization other than soil properties, such as the urgency of construction, the location of the stabilized layer in the pavement, the type of construction equipment available or needed, and the influence of environmental conditions on the stabiliz [illegible] layer.

SSIS was to be developed in two separate and consecutive phases. Phase One was the establishment of the system based entirely on existing knowledge and experience; no physical testing was accomplished. The important references in worldwide literature were read and over 40 acknowledged experts in the soil stabilization field were contacted for this phase of the research. The total system is somewhat lengthy and it is not appropriate to repeat the entire system at this point. However, a brief discussion of the system and some of the salient charts are necessary to set the stage for the ensuing portions of the report.

Figure 1 is an overview of the total system. It should be noted that

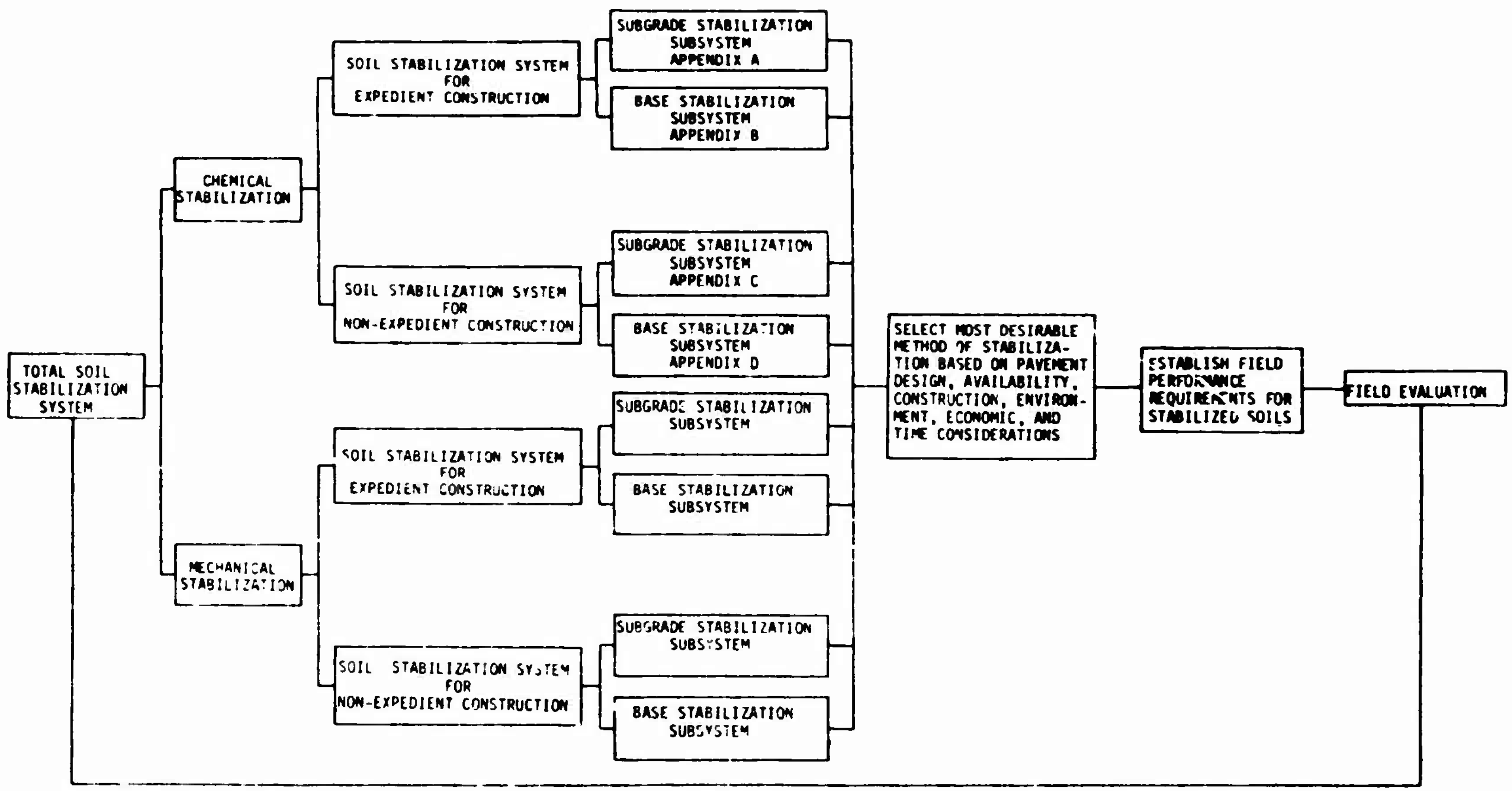


Figure 1 - The Air Force Soil Stabilization System (from reference 4)

(NOTE: Appendix Nos. refer to those in original reference)

the system is divided into two main divisions: chemical and mechanical stabilization. Within each of these divisions there are systems for both expedient and nonexpedient construction to recognize the Air Force requirements for situations where time, availability of construction equipment and many other factors dictate the type of effort that will be put into the stabilization project. Expedient construction demands rapid testing and evaluation if chemical admixtures are to be used, it means that some of the stringent requirements that ensure long-term performance must be relaxed, and it often means that economic considerations are not foremost. Nonexpedient construction, on the other hand, has the connotation of all that is expected from permanent construction.

The expedient and nonexpedient systems are each broken down to subsystems for subgrade (Figures 2 and 4) and base (Figures 3 and 5). Subbases are not explicitly mentioned, but they may fit into either of the other two subsystems depending on whether the material is predominantly granular (base-like) or fine grained (subgrade-like). Figures 2 through 5 show that the initial breakdown for determining type of stabilizer (lime, cement or asphalt) to be added to a soil is based on gradation -- more precisely, on the percent passing the No. 200 sieve -- and on the plasticity characteristics of the fines in the soil as described by the plasticity index. Both of these parameters are used to distinguish between soils in all major soil classification schemes, and while they do not, by any means, completely describe a soil's suitability for stabilization, they indirectly indicate such factors as ease of mixing and physico-chemical properties that may influence stabilization of a soil. However, they are not infallible indicators and it is necessary to perform additional tests to determine the amount of the selected stabilizer to add and to evaluate its effectiveness. In each case, this is performed by following

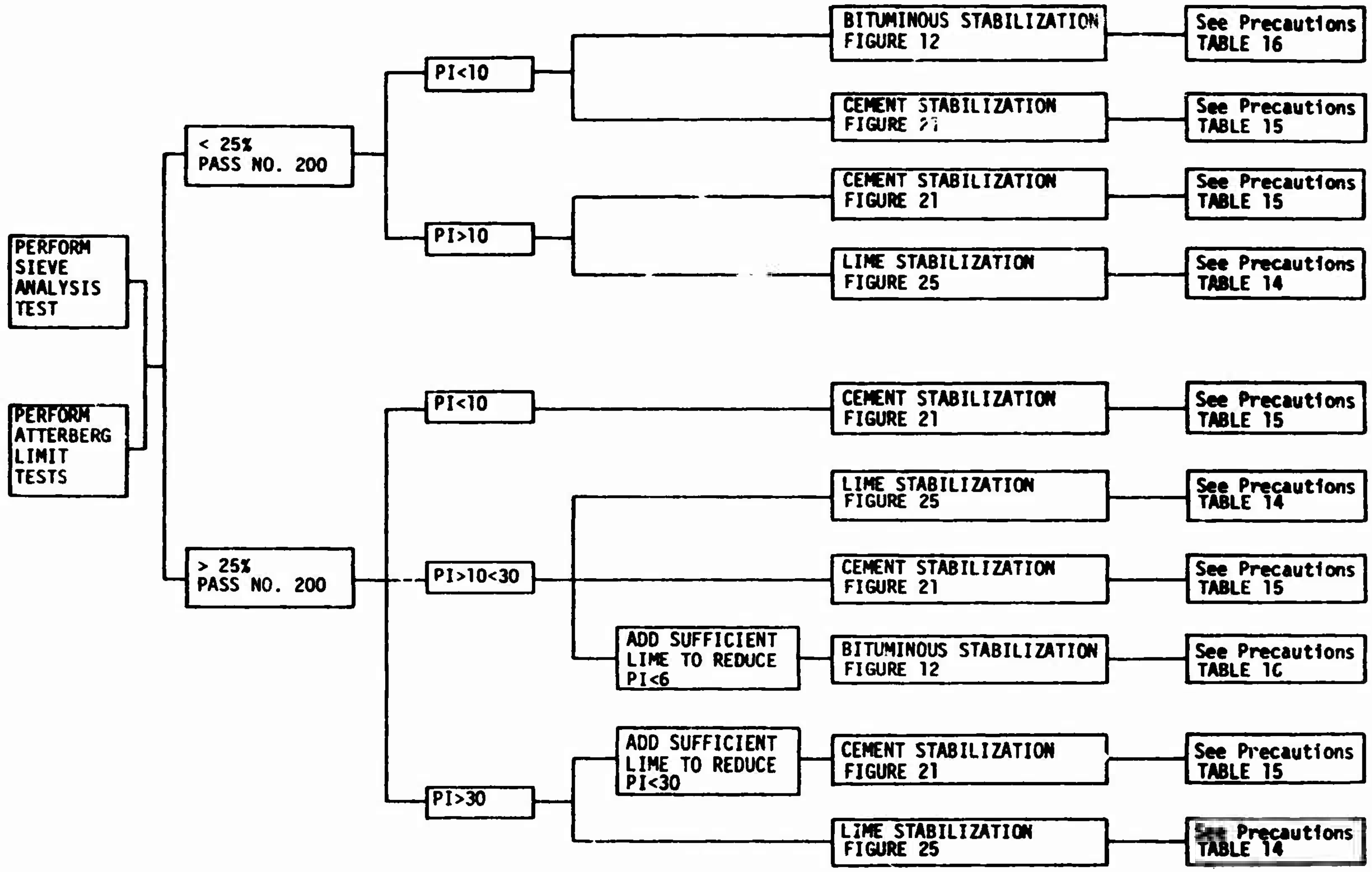


Figure 2 - Selection of Stabilizer for Expedient Subgrade Construction (from reference 4)

(NOTE: Figure and Table Nos. refer to those in original reference)

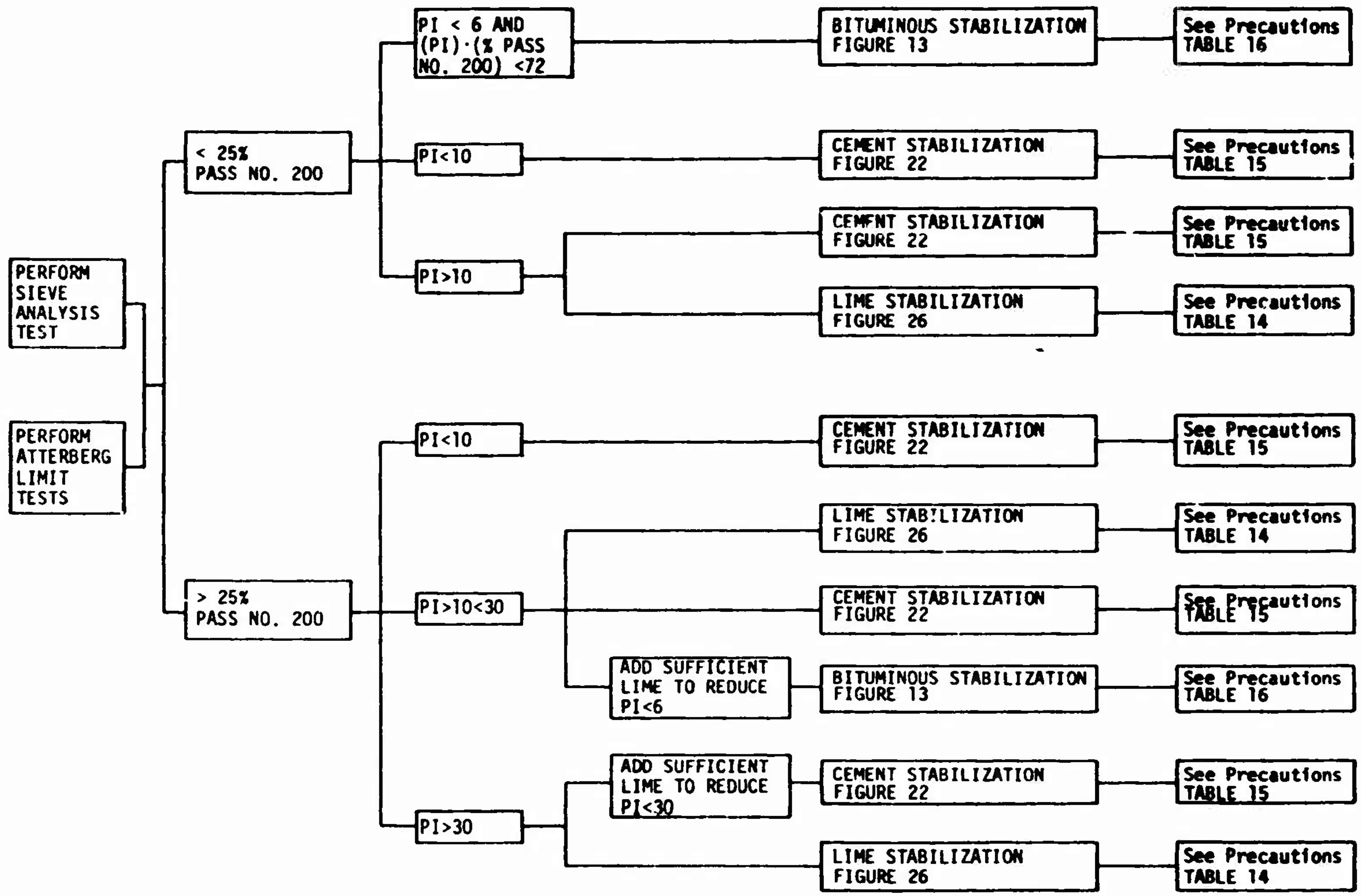


Figure 3 - Selection of Stabilizer for Expedient Base Construction (from reference 4)

(NOTE: Figure and Table Nos. refer to those in original reference)

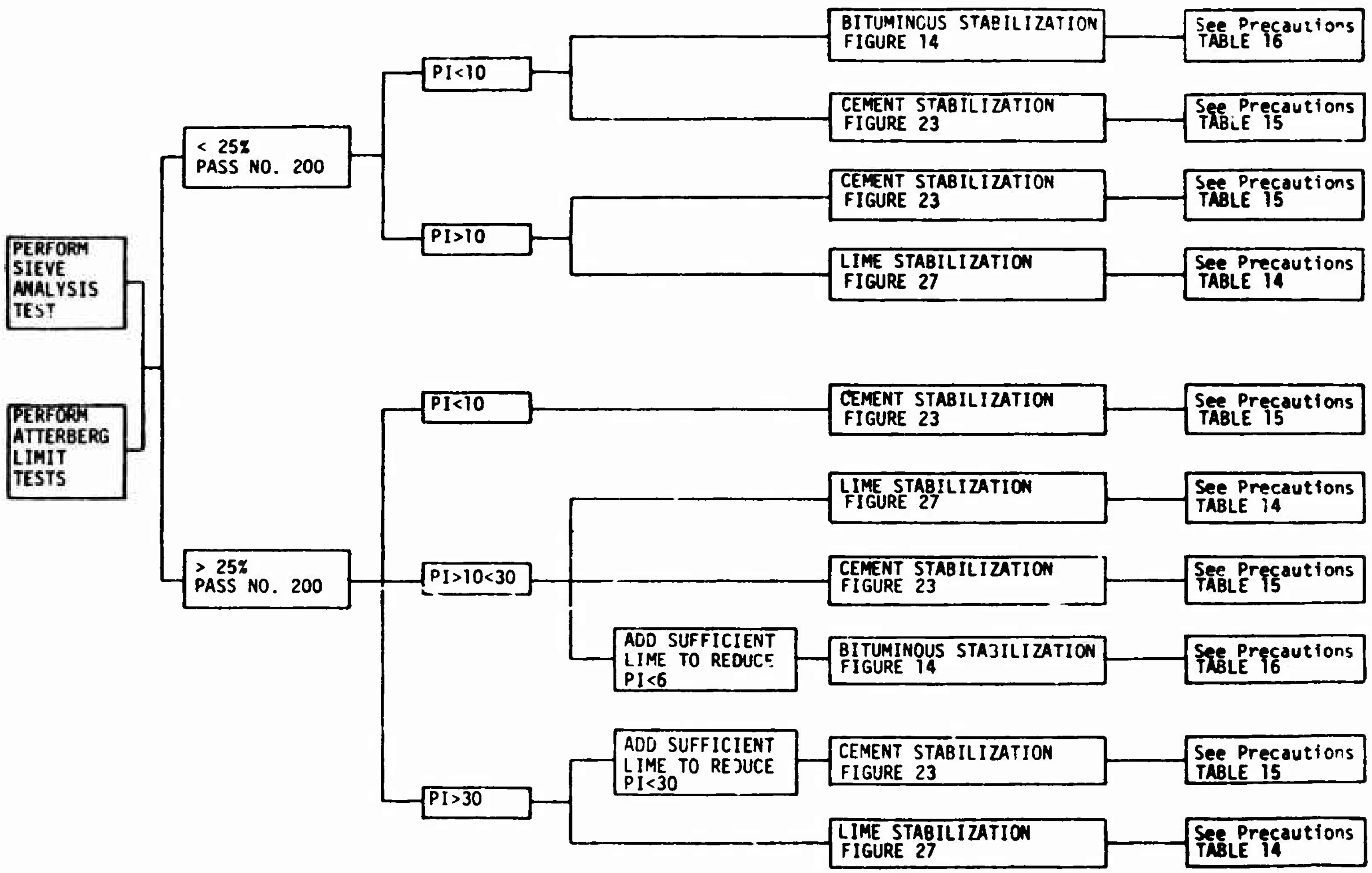


Figure 4 - Selection of Stabilizer for Nonexpedient Subgrade Construction (from reference 4)

(NOTE: Figure and Table Nos. refer to those in original reference)

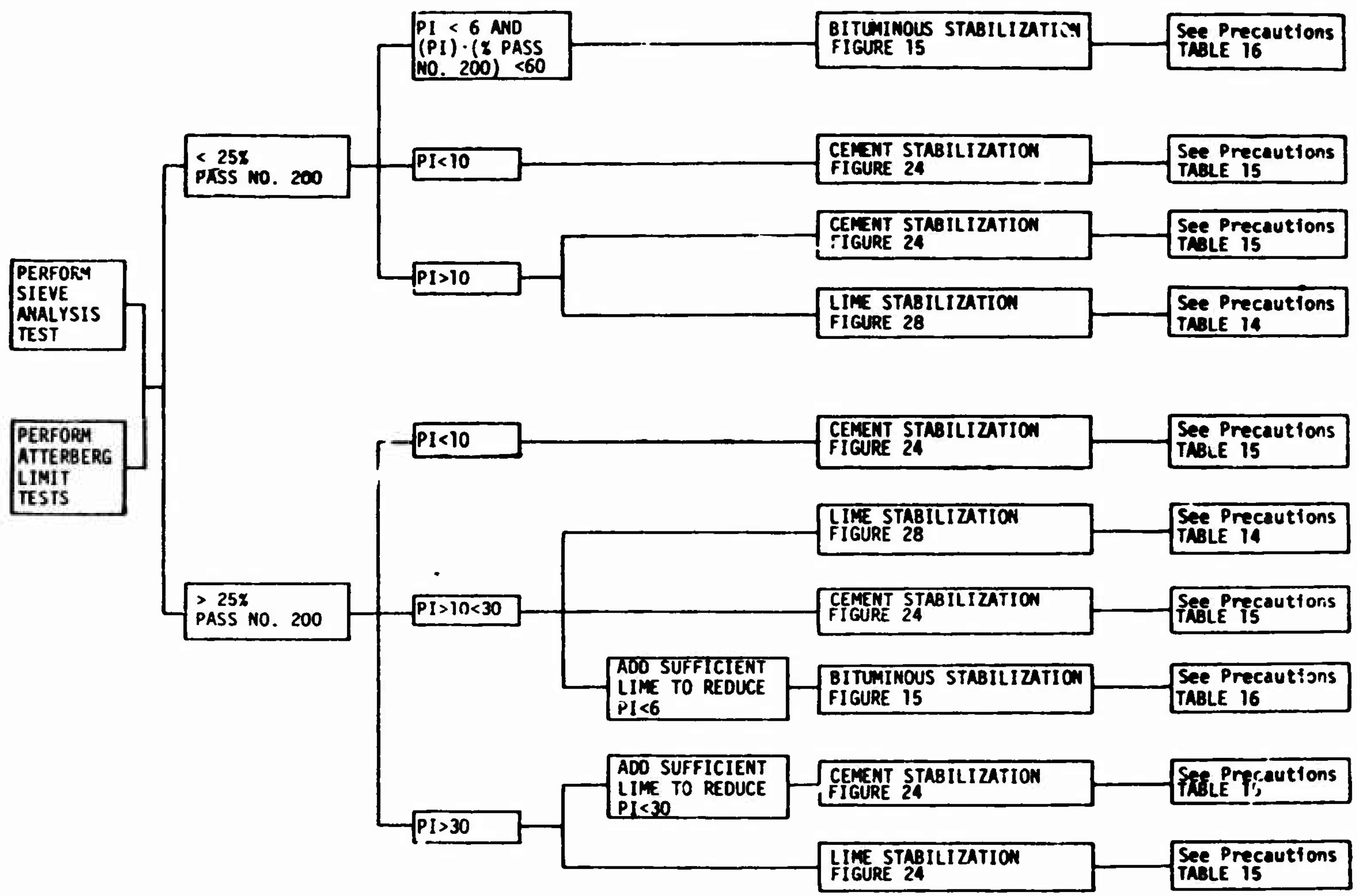


Figure 5 - Selection of Stabilizer for Nonexpedient Base Construction (from reference 4)

(NOTE: Figure and Table Nos. refer to those in original references)

additional steps which are given in the various subsystems, all of which lead to a final evaluation of the soil-stabilizer system. These separate subsystems are not presented at this point in the report, but they will be discussed later.

Not all soils are capable of being satisfactorily stabilized with the three stabilizers utilized in the SSIS. However, some engineering properties of nearly every soil can be improved. The distinction between stabilization, implying a major improvement in strength, and modification, which denotes improvement in engineering behavior without appreciable gains in strength, was discussed in the development of SSIS (reference 4), but it was apparent that the distinction between the two was one which is difficult to make. For this reason, SSIS was not developed into separate systems for stabilization and modification.

In the final analysis, the development of SSIS indicated that there were many gaps in knowledge in the area of soil stabilization. Some of the questions which remain may never be answered owing to the many variations in soils which Nature has imposed on us. Other problems, such as the effect of organic compounds and salts in the soils on their stabilization potential, have received attention, but definitive criteria were not available during development of SSIS. An attempt to fill some of these gaps was the purpose of Phase Two, or the validation of SSIS. This was to be done primarily by laboratory testing of soils and, to the extent possible, by discussion of the system with various authorities in the soil stabilization field. Fourteen separate soil groups were identified in SSIS, based on the percent passing the No. 200 sieve, plasticity index, sulfate content, and organic content. These are shown below as soil groups:

Group No.	Percent Passing No. 200 Sieve	Plasticity Index	Sulfate Content	Organic Content
1	>25	>30	high	low
2	>25	>30	low	low
3	>25	>30	high	high
4	>25	>30	low	high
5	>25	>10<30	high	low
6	>25	>10<30	low	low
7	>25	>10<30	high	high
8	>25	>10<30	low	high
9	<25	<6	low	high
10	<25	<6	low	low
11	<25	Non-plastic	low	low
12	<25	>10	low	low
13	<25	<10	low	low
14	>25	<10	low	low

An attempt was to be made to locate at least one soil from each of these groups and test it to determine the validity of SSIS in Phase Two. Based on the test results and discussion with various authorities, SSIS was to be revised, if necessary.

3. Scope of Report

This report is concerned primarily with the validation of the index system, as described above, through the use of laboratory testing and discussion of the present SSIS with authorities in the soil stabilization field. Also included are the results of repeated loading tests on selected stabilized

soils which were molded into beams and diaphragms for testing purposes. Additional study beyond the SSIS verification aspects, was conducted concerning the use of the pH test developed by Eades and Grim (reference 5) for estimating the optimum lime content in soils, and in reviewing the criteria for soil cement stabilization.

The mechanical stabilization subsystems, although included in the overall SSIS shown in Figure 1, were not under consideration in this project, and they are not included in this report.

SECTION II

MATERIALS UTILIZED

1. General

Twenty-two soils were collected for use in the validation phase. They are designated by the location where they were obtained. Air Force personnel collected several of the soils from both active and disused U.S. Air Force bases. The remaining soils were collected by project personnel to complete the desired groups of soils mentioned earlier.

All soils were treated in essentially the same manner after they were received in the laboratory. The several containers of each soil were combined and a representative sample was obtained to perform the following classification tests: grain size, Atterberg limits, pH, sulfate content and organic content. If the soil was deemed suitable by virtue of meeting the requirements for one of the 14 desired groups, the entire amount of soil was then air dried (if organic) or oven dried at a temperature not exceeding 120°F (inorganic soils). Fine grained soils were then pulverized to pass a 1/8 inch sieve and coarse grained soils were sieved into several different size ranges so they could be recombined into the desired gradations.

The prepared soils were sampled again, and the above mentioned tests were repeated; the latter test values are reported in Table 1. In addition, compaction and strength characteristics were determined, and the soil was classified.

2. Brief Description of Soils

The soils used during the research are described below.

Tuy Hoa was a poorly graded beach sand from Tuy Hoa, Vietnam, which con-

TABLE 1

PROPERTIES OF SOILS

Item	SOIL								
	Tuy Hoa	Altus Subbase	Dyess	Altus Subgrade	Tyler	Houma	Perrin B	Perrin A	Perrin AB
Color		Reddish Brown	Reddish Brown	Reddish Brown	Black	Dark Grey	Light Yellow	Light Grey	--
Physical Properties:									
Liquid limit	--	14.5	40.3	40.7	52.5	61.7	65.0	72.0	69.4
Plasticity index	N.P.[a]	N.P.	23.2	19.8	21.1	40.8	41.7	40.0	43.3
Specific gravity	2.65	2.66	2.69	2.79	2.67	2.69	2.84	2.74	2.78
Max. Dry density, pcf[b]	--	--	102.7	97.7	91.7	36.4	92.4	97.5	95.0
Opt. moist. content, %[b]	--	--	19.7	23.6	22.3	23.7	24.1	23.7	23.9
Unconfined compressive strength, psi[c]	--	--	29.3	36.5	39.6	36.0	53.4	37.3	34.1
Chemical Properties:									
Sulfate (as SO_4), %[d]	0.00	0.09	0.00	1.52	3.50	0.00	7.64	1.00	4.30
Organic matter, %[e]	3.54	0.15	1.17	0.07	3.00	1.43	0.13	0.00	--
pH (1:1)[f]	5.10	7.40	7.40	7.50	2.30	6.95	7.30	4.50	6.70
Grain Size Analysis:									
% passing No. 4	100	100	100	100	100	100	100	100	100
10	97.0	100	100	100	100	100	100	100	100
20	58.5	98.0	99.0	100	100	100	100	100	100
40	12.0	94.0	95.0	93.0	100	100	100	100	100
100	3.0	30.0	73.5	97.0	100	98.0	97.5	98.0	98.0
200	2.5	15.0	67.5	91.0	92.5	92.5	96.5	95.0	97.0
-2.	--	7.0	37.0	61.5	52.5	58.0	73.0	70.0	71.0
Engineering Classification.									
AASHO	A-1-b	A-2-4	A-7-6(12)	A-7-6(12)	A-7-5(15)	A-7-6(20)	A-7-6(20)	A-7-6(20)	A-7-6(20)
Unified	--9	SC	CL	CL	OH	CH	CH	CH	CH

TABLE 1 (CONTINUED)

Item	Soil: Panama A	Panama B	North Carolina	Big Brown	Brazos	Bryan	Dansby	Yoakum	Dallas Regional	WES Gravelly Sand	WES Lean Clay	WES Buck-Shot Clay	Chenault
Color	Red and Light Gray	Red and Light Gray	Reddish Brown						Dark Green				
Physical Properties:													
Liquid Limit	72.5	75.5	61.0	N.P.	N.P.	N.P.	N.P.	N.P.	68	25	37.5	67.1	45.6
Plasticity index	32.8	35.5	26.9	N.P.	N.P.	N.P.	N.P.	N.P.	50	3	13.6	43.0	29.6
Specific gravity	--	--	--	2.62	--	--	--	--	--	2.60	2.72	--	--
Maximum dry density p4[b]	83.4	82.8	98.6	--	111.8	--	--	118.5	--	132.7	107.3	--	--
Opt. moist. content, %[b]	35.1	35.1	23.5	--	7.8	--	--	11.5	--	7.8	17.8	--	--
Unconfined compressive Strength, psi[c]													
Chemical properties:													
Sulfate (as SO_4), %[d]									0.05		0.004	0.005	0.008
Organic matter, %[e]									0.67		0.23	0.96	0.28
pH (1:1)[f]	5.30	6.27	5.05						7.73		--	--	7.70
Grain Size Analysis													
% passing No. 3/4"				98	--	100	--	--		100			
1/2"				97	--	98	100	--		85			
3/8"				93	--	91	99	--		81			
4				83	68	53	98	56		66			
10				73	39	38	97	46		61			
20				65	29	28	95	38		53			
40				61	18	17	93	34		35			
100				21	3	6	30	12		1			
200				8	1	1	6	2	38		96	94	75
-2µ											17		
Engineering Classification													
AASHO									A-7-6(20)		A-6(9)	A-7-6(20)	A-7-6(17)
Unified	CH	CH	CH						CH		CL	CH	CH

[a]Nonplastic.

[b]AASHO Designation: T 99-70

[c]Determined on 2 in. diameter by 4 in. high specimens compacted at optimum water content and ± 2 pcf of AASHO T 99-70 density.

[d]Using turbidimetric method

[e]Using wet combustion method (AASHO Designation: T 194-70).

[f]Equal parts of soil and water are used to determine soil pH.

[g]Fine fraction contains predominantly organic matter with little, if any, clay. Therefore, the soil was not classified according to Unified Classification System.

tained a large percentage of plant roots. It had the appearance of being quite dusty; however, the majority of the material passing the No. 40 sieve was plant roots which could easily be removed by washing. This soil was acidic (pH = 5.10). It was selected primarily because of the reports that portland cement stabilization had been attempted with unsuccessful results. Sampling procedures and precise location are unknown, so it is not certain that this was the exact material utilized in the cement stabilization project. Certainly, it must be very similar. This is a soil difficult to classify in any of the standard soil classification schemes. It did not contain enough organic material to be denoted a peat but a Unified Soil Classification of SP belies its true nature.

Altus Subbase, from Altus Air Force Base, Oklahoma, was a red-brown fine sand containing a small percentage of fines. The soil was slightly alkaline. It was collected by Air Force personnel from the subbase of a runway during reconstruction.

Dyess was from Dyess Air Force Base, Taylor County, Texas. This was a reddish-brown and mildly alkaline silty clay containing some organic matter and a small amount of calcium carbonate.

Altus Subgrade was also from Altus Air Force Base, Oklahoma. This was a reddish-brown and mildly alkaline plastic clay containing gypsum and iron. It was obtained from a runway subgrade during reconstruction.

Houma was a dark gray plastic clay obtained from Houma, Louisiana. The general area has been used as a test site by the Air Force. The soil has a high organic content, but it is nearly neutral. Montmorillonite is the predominant clay mineral in the soil (reference 6).

Perrin A and Perrin B were two soils collected from Perrin Air Force Base, Grayson County, Texas. It had been reported that during the construction

of a runway on this soil unexpected problems occurred with the use of lime as a stabilizer due to a high gypsum content (reference 7). Standard laboratory tests conducted prior to construction did not detect this condition. Following directions regarding its location, Air Force personnel collected a large quantity of the soil which was permeated with thin layers and streaks of a yellow powder between the bedding planes of this shale-like clay. Subsequent tests indicated that this soil, designated Perrin A, contained only 1.0 percent sulfate. A second soil, termed Perrin B, was obtained from another location on the airfield, and this proved to have a sulfate content of 7.64 percent. Perrin A was a light gray and acidic plastic clay, whereas Perrin B was a light yellow and mildly alkaline plastic clay. Both soils were very hard when air dried and were very soft and swelled when soaked in water. They had the same mineralogical content except for the difference in gypsum content. An artificial mixture of equal parts of Perrin A and Perrin B, designated Perrin AB, was also used for the study. This presented an excellent opportunity to study the effect of sulfates on the stabilization potential of the clay.

Tyler was a black, medium plastic clay, obtained from a cut on the side of Interstate Highway 20 north of Tyler, Texas. This was a very acidic soil (pH = 2.3) containing both high sulfate and organic contents. It has been the subject of considerable study in another field of endeavor because of the inability to sustain vegetation on it where it served as the backslope of the highway (reference 8). It was the only soil located which fell into the Group 7 of the soils desired for validating the system.

WES Gravelly Sand, WES Lean Clay and WES Buckshot Clay were all obtained from the Waterways Experiment Station, Vicksburg Mississippi where they were used as elements in the MWHGL (multiwheel heavy gear load) test section (reference 9). WES Lean Clay is a low plasticity, gray clay with low sulfate and

organic content. WES Buckshot Clay was a high plasticity dark gray clay with low sulfate content but nearly one percent organic matter. The Gravelly Sand had low organic and sulfate contents. The gradation and plasticity of the latter material did not coincide with that reportedly used in the test section. After obtaining a new shipment of the material, it was finally ascertained that the original stockpile had been used in the test section construction and a new stockpile was being utilized for the samples shipped to Texas A&M University.

Dallas Regional was a subgrade material at the Dallas-Ft. Worth Regional Airport. The primary purpose for obtaining this material was that previous research (reference 2) indicated it was virtually impossible to stabilize, probably owing to its high organic content. The sample whose properties are reported in Table 1 was obtained more than a year after the previous research sample was taken, and probably at a different location, since the exact location of the first sample was not recorded. The organic content, as seen in Table 1, was not extraordinarily high and the sample received only limited use.

Chenault was obtained from Chenault AFB, Louisiana. It was a dark gray silty clay and received only limited use since it fell in a group in which a suitable material had already been located.

Two soils, designated Panama A and Panama B, were supplied by the U.S. Air Force. Panama A was obtained from Albrook AFB and Panama B from Howard AFB, both in the Canal Zone. Physical properties of the two soils were very similar, but their chemical properties varied. Both soils were acid, and both had a red and light gray mottled appearance. Certain characteristics of these soils varied depending on the degree of pulverization they underwent during preparation.

North Carolina was obtained by Air Force personnel from Catawba County, North Carolina. It was a friable, acid soil with reddish brown color.

North Carolina and the two Panama soils were selected as they were lateritic or lateritic-like soils. Previous research (reference 36) indicated some peculiarities with these soils regarding stabilization, and it was desired to determine whether SSIS would detect any problems. All three soils were received in essentially an air-dried condition.

The remaining soils, Bryan, Brazos, Yoakum, East Texas and Dansby received use either in the verification of the bituminous stabilization subsystem (Section V) or in dynamic testing studies of stabilized materials (Section VI). They were all obtained from Central or East Texas.

SECTION III

LIME STABILIZATION

1. Present System and Research Needs

In SSIS, the decision as to whether lime should be used as a stabilizing agent for a soil is based on the plasticity index of the material passing the No. 40 sieve (see Figures 2, 3, 4 and 5). If the plasticity index exceeds 10, the soil is considered a likely candidate for lime stabilization and one of the subsystems is entered (Figure 6, 7, 8 or 9) depending on the use of material. Each subsystem represented by Figures 6 through 9 utilizes the pH test advocated by Eades and Grim (reference 5) for estimating the optimum lime content. This estimated optimum is utilized without modification for the expedient subgrade situation, whereas for the remaining situations it is utilized as a starting point for additional investigations to determine: a) whether the soil is truly "lime reactive," b) the optimum lime content based on strength considerations, and c) the durability of the lime stabilized soil. The importance of the pH test warranted additional research to determine its accuracy with respect to SSIS and to determine whether it could or should be replaced. The effects of organics and sulfates on lime stabilization were alluded to by inclusion of Table 2 as proposed by Thompson (reference 10). This is basically a durability criterion which requires certain unconfined compressive strengths after freezing and thawing cycles or after soaking. This also deserved attention in the validation phase, with particular emphasis needed on means of hastening durability testing so it might be included when necessary in the expedient aspect of the system. Finally, a means of reducing the quantity of soil required for testing was considered important, as large test samples mean more time spent in preparing larger quantities of soil for testing.

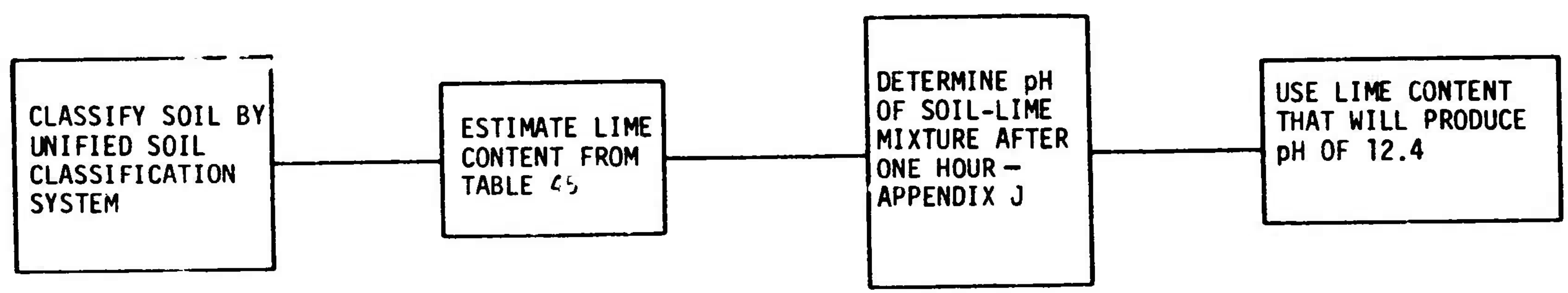


Figure 6 - Subsystem for Expedient Subgrade Stabilization With Lime (from reference 4)

(NOTE: Table and Appendix Nos. refer to those in original reference)

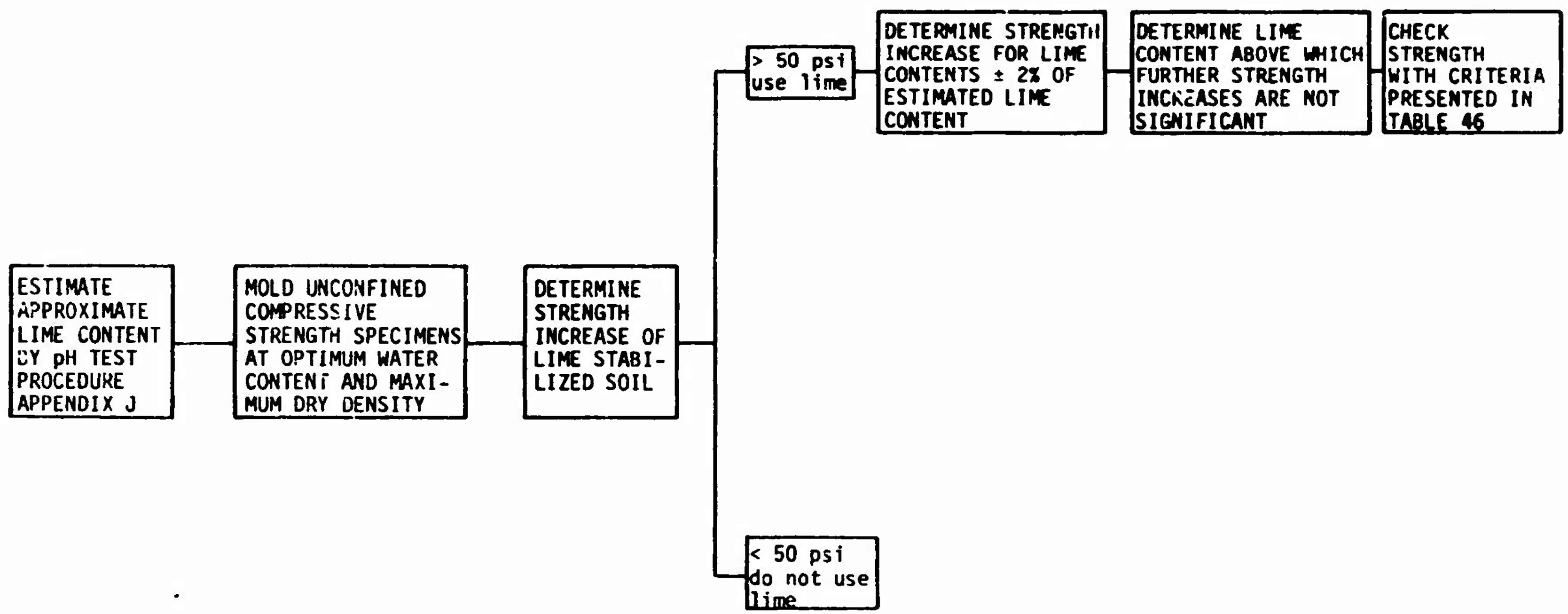


Figure 7 - Subsystem for Expedient Base Course Stabilization With Lime (from reference 4)

(NOTE: Table and Appendix Nos. refer to those in original reference)

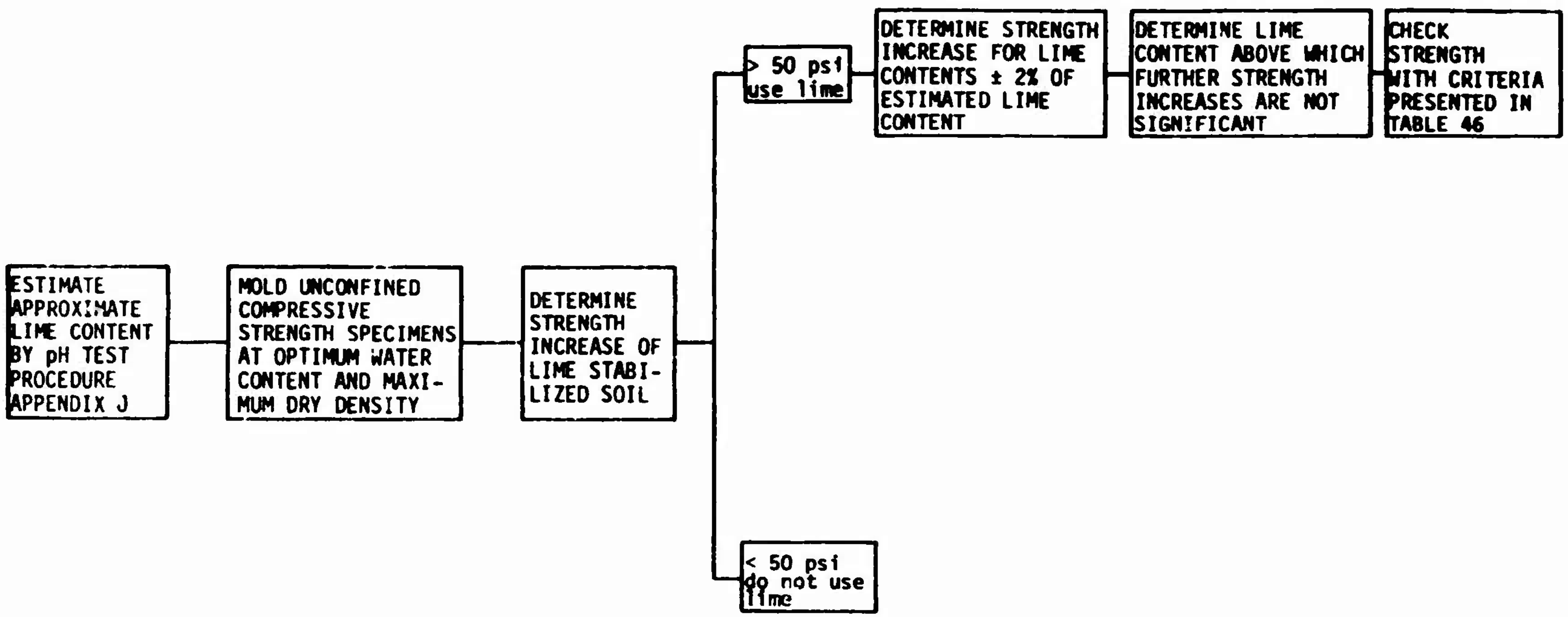


Figure 8 - Subsystem for Nonexpedient Subgrade Stabilization With Lime (from reference 4)

(NOTE: Table and Appendix Nos. refer to those in original reference)

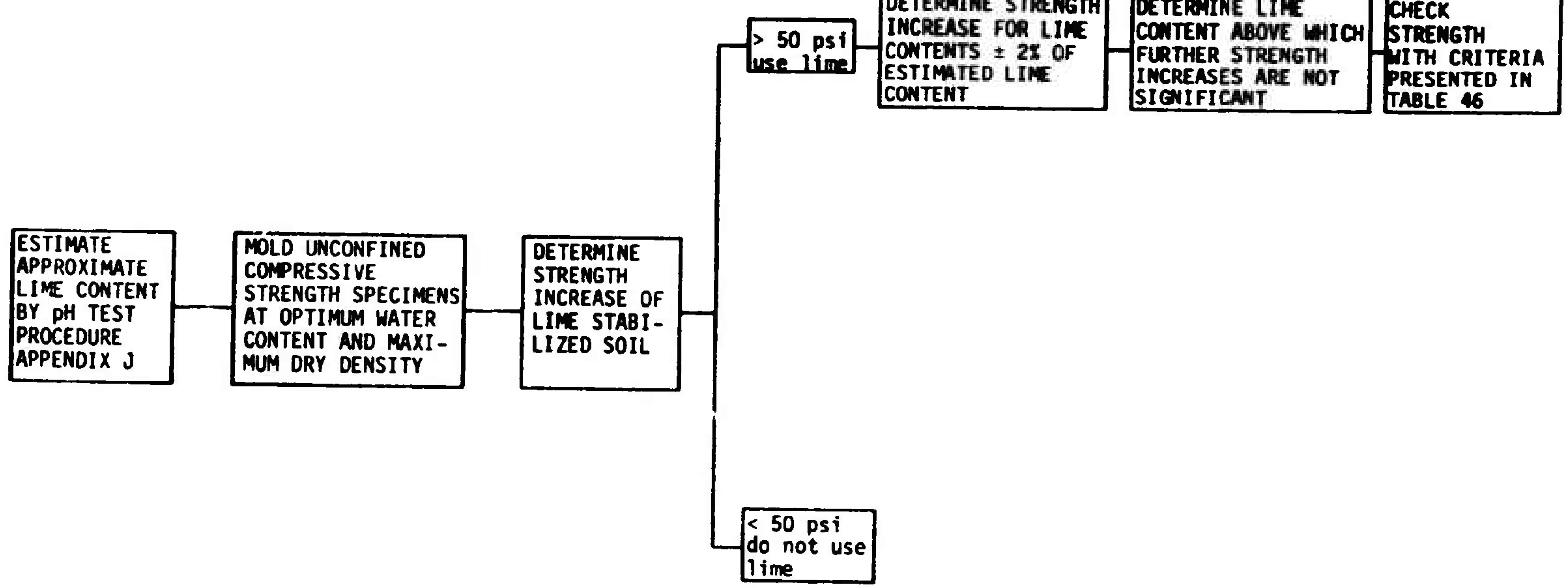


Figure 9 - Subsystem for Nonexpedient Base Course Stabilization With Lime (from reference 4)

(NOTE: Table and Appendix Nos. refer to those in original reference)

TABLE 2

TENTATIVE LIME-SOIL MIXTURE COMPRESSIVE STRENGTH REQUIREMENTS

Anticipated Use	Residual Strength Requirement, psi (a)	Strength Requirements for Various Anticipated Service Conditions (b)			
		Extended (8 day) Soaking (psi)	Cyclic Freeze-Thaw (e) 3 Cycles (psi)	7 Cycles (psi)	10 Cycles (psi)
Modified Subgrade	20	50	50	90 50*	120
Subbase					
Rigid Pavement	20	50	50	90 50*	120
Flexible Pavement					
Thickness of Cover (c)					
10 inches	30	60	60	100 60*	130
8 inches	40	70	70	110 75*	140
5 inches	60	90	90	150 100*	160
Base	100(d)	130	130	170 150*	200

a) Minimum anticipated strength following first winter exposure.

b) Strength required at termination of field curing (following construction) to provide adequate residual strength.

c) Total pavement thickness overlying the subbase. The requirements are based on the Boussinesq stress distribution. Rigid pavement requirements apply if cemented materials are used as base courses.

d) Flexural strength should be considered in thickness design.

e) Number of freeze-thaw cycles expected in the lime-soil layer during the first winter of service.

*Note: Freeze-thaw strength losses based on 10 psi/cycle except for 7 cycle values indicated by an * which were based on a previously established regression equation.

(1 reference 10)

?. The pH Test

For many years it was considered that soils were stabilized with lime by virtue of cation exchange, cementation from the formation of calcium carbonate, and the rather elusive pozzolanic reaction. As a result of research, much of it conducted at the University of Illinois by Eades and Grim (reference 11), it is now realized that the strength-producing properties are primarily the result of the dissolution (or partial dissolution) of clay minerals in the highly alkaline environment produced by lime, and the recombining of the aluminum and silica from the clay with the calcium from the lime to form complex calcium aluminum silicates which cement the remaining grains together. The purpose of the pH test developed by Eades and Grim is to determine how much lime must be added to a soil to create and maintain the high pH necessary to dissolve the clay minerals. However, the high pH alone will not ensure stabilization if reactive minerals are not present. Also, as the lime is utilized in the reaction process, the pH will decrease, so the amount of lime to just obtain a pH of 12.4 may not be sufficient to maintain this condition over a lengthy period if it is desired to sustain the stabilization reaction.

Table 3 shows the results of pH tests performed according to the procedure outlined by Eades and Grim. A pH meter with an accuracy of 0.001 was utilized; however, the readings are rounded off to the nearest 0.01. Some of the interpretive difficulties which the novice might face are apparent from the table. It would be particularly desirable to know how close one must come to a pH of 12.4 to expect a satisfactory reaction, particularly if a logistic situation existed such that there was not an ample supply of lime. The interpretation of the pH test will be discussed in detail later in this section.

TABLE 3

pH (1:5) OF SOIL-LIME MIXTURES AFTER ONE HOUR

Percent Lime	Soils						
	Perrin B	Perrin A	Perrin AB	Altus Subgrade	Dyess	Houma	Tyler
0	8.00	5.30	7.30	8.04	8.46	7.92	2.84
1				11.74	11.93	11.27	
2	12.20			12.17	12.28	12.07	
3	12.33	12.12	12.23	12.32	12.36	12.23	
4	12.35			12.35	12.40	12.37	
5	12.37	12.30	12.31	12.40	12.40	12.40	9.89
6	12.40			12.40		12.40	10.54
7	12.40	12.37	12.36				11.07
8		12.40	12.40				11.59
10							12.10
12							12.40
14							12.40

3. Lime Influence on Soil Plasticity

It has been suggested that the Atterberg limits can also be used to determine the optimum amount of lime to add to a soil to achieve stabilization. Atterberg limit tests were performed on soil-lime mixtures (for seven soils) containing varying amounts of lime, and these results (Figures 10 through 13) were examined to determine the optimum lime content or "lime fixation" percentage. This optimum was compared to that determined by the pH test.

The test procedure involved mixing the dry soil and lime together, adding distilled water to obtain roughly the liquid limit of the mixture, and storing the mixture in a covered porcelain dish for 24 hours before the test was performed.

The liquid limit results were not consistent for all soils, as both increases and decreases were observed with increased lime percentages. However, the plasticity characteristics of all soils were improved to varying degrees by the lime, and workability of all soils was greatly improved. The plasticity indices of the Dyess and Houma mixtures were considerably reduced by lime (Figure 10). The plasticity index of the Tyler soil was not further reduced after the addition of 2 percent lime (Figure 11), and the plasticity indices of the high-sulfate content soils, Perrin A, Perrin B, Perrin AB and Altus subgrade did not appreciably decrease for the lime contents used in the investigation (Figures 12 and 13).

A graphical comparison of the relationship between the optimum lime content as determined by the plasticity index and by the pH test is shown in Figure 14. Except for the Tyler soil, the two tests gave results which compared closely.

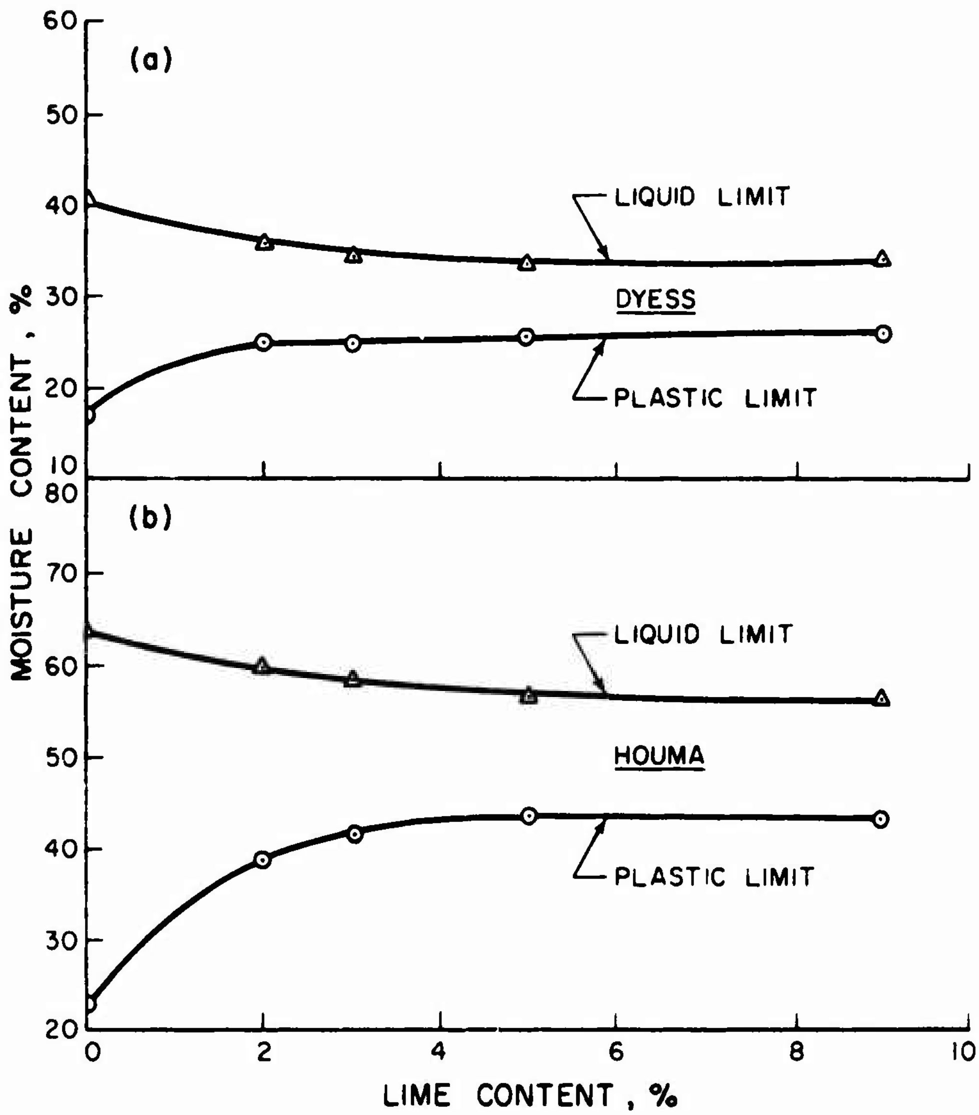


Figure 10 - Effect of Lime on Atterberg Limits of Dyess and Houma Soils

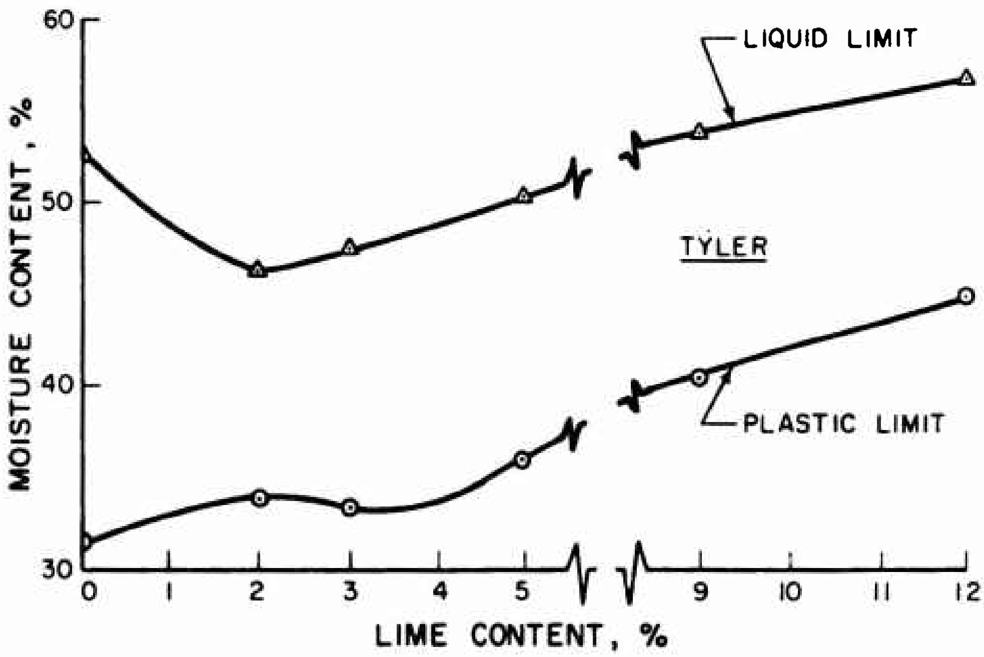


Figure 11 - Effect of Lime on Atterberg Limits of Tyler Soil

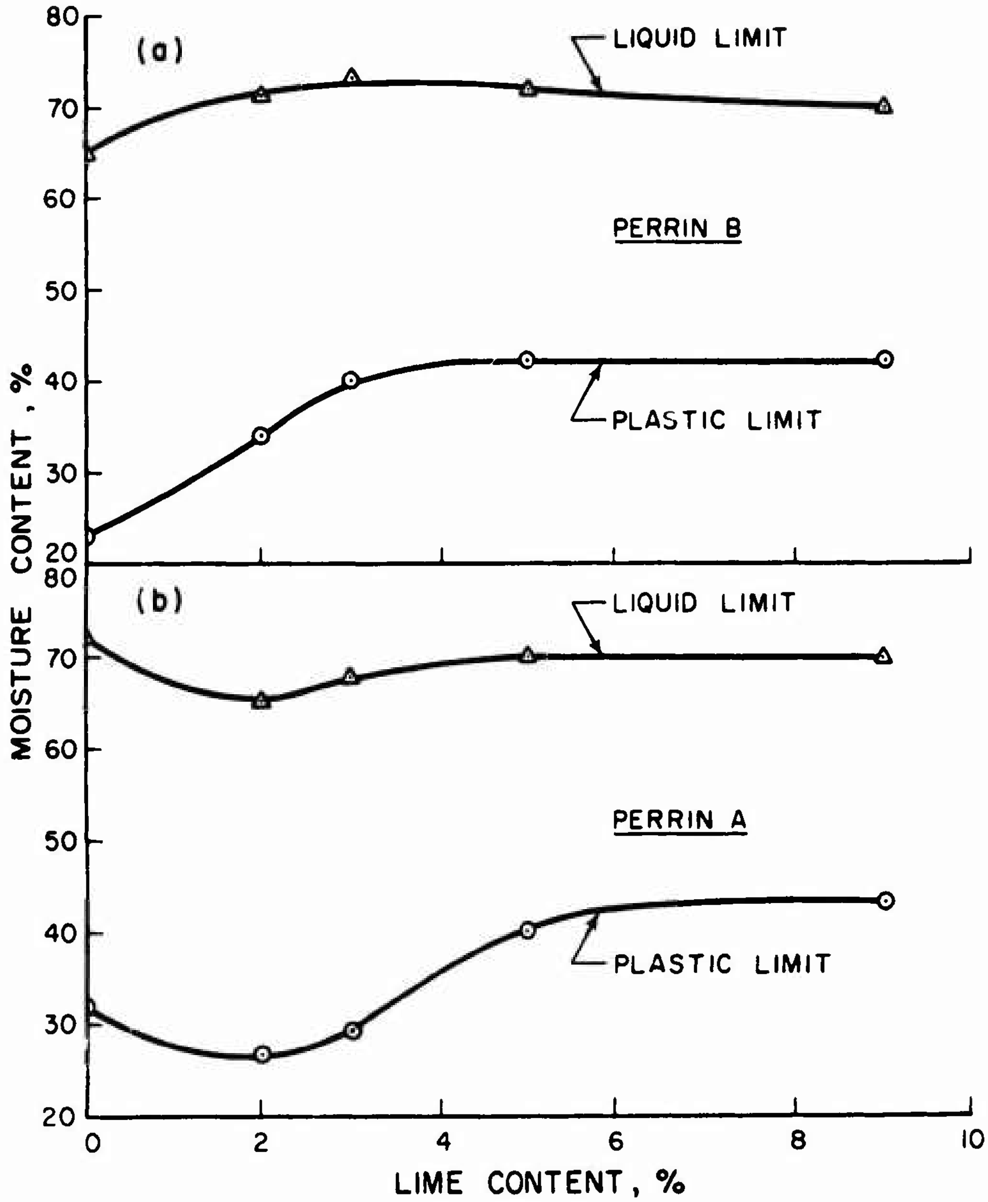


Figure 12 - Effect of Lime on Atterberg Limits of Perrin B and Perrin A Soils

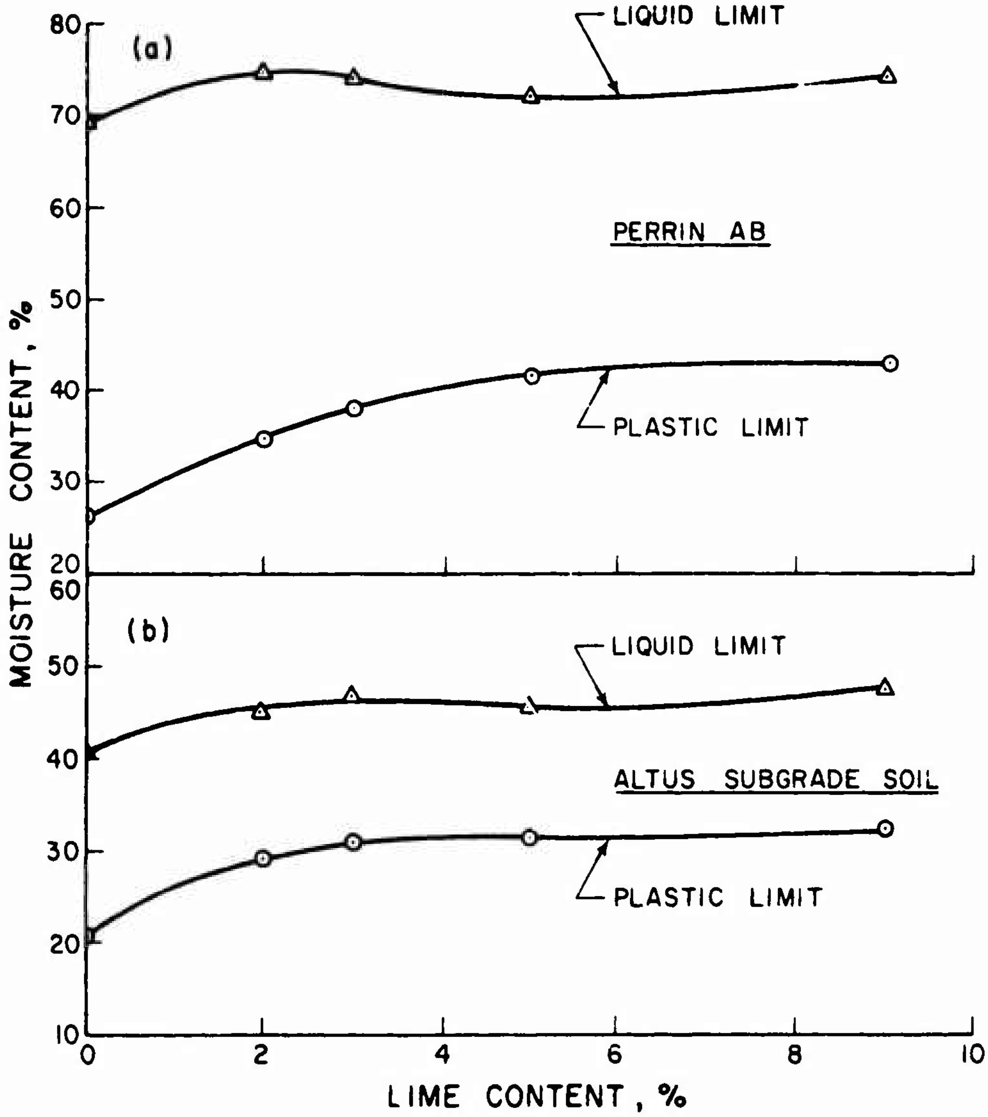


Figure 13 - Effect of Lime on Atterberg Limits of Perrin AB and Altus Subgrade Soils

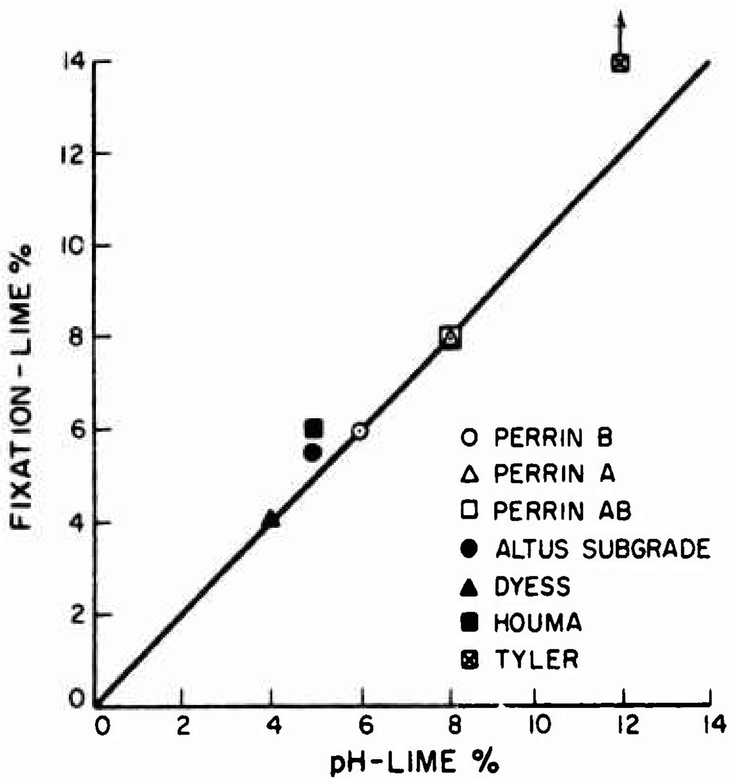


Figure 14 - Relation Between Fixation-Lime Percentage and pH-Lime Percentage

4. Preparation of Specimens for Strength and Durability Tests

a. Selection of compaction technique.

As mentioned earlier, the selection of sample size for strength and durability tests, as well as other tests, poses a problem for field use. This is particularly true in the case of expedient operations where the time available for preparation of soil may be limited. Most soil compaction procedures utilize the 1/30 cu. ft. Standard AASHO mold or the larger CBR mold. However, neither of these molds produces samples which are suitable for compression tests unless they are first trimmed. Some investigators have used the Harvard miniature compactor for soil stabilization studies, but this is not a standard item of equipment for any of the U.S. Military services. Two-inch diameter by 2-inch high specimens have been utilized in some soil cement studies and there are several studies where 2-inch diameter by 4-inch high specimens have been used (reference 12). The latter are much more suitable for compression tests and it also appears that equipment for forming samples of this size can be fashioned from the mold and hammer available in the Soil Trafficability Kit developed by the U.S. Army Corps of Engineers (reference 13). The device known as the Vicksburg miniature compactor (Figure 15) also produces 2- x 4-inch specimens. This apparatus uses a 5-pound hammer to compact the soil, generally in 4 layers, each 1-inch thick. Usually, 4 or 5 tamps will reproduce the Standard AASHO compactive effort, although a correlation should be obtained for each soil. After compaction, the sample height is measured to the nearest 0.001-inch with a dial gage comparator (this allows easy computation of the unit weight of the sample), and the sample is extruded from the mold. As a result of its ease of use and applicability to the problem at hand, the Vicksburg miniature compactor was selected to produce specimens for

Figure 15 - Vicksburg Miniature Compaction Apparatus Used for Molding 2-Inch Diameter by 4-Inch High Specimens

strength and durability testing in the lime verification program. However, comparison compaction tests were also performed to determine the optimum moisture content and maximum unit weight in accordance with AASHO Designation T-99-70 (reference 14).

b. Preparation of mixture.

Air-dried soil passing the No. 10 sieve was dry mixed with lime for 2 minutes in a Lancaster counter current batch mixer. Sufficient water was then added to bring the mixture to the desired moisture content and mixing was continued for 2 more minutes. The materials were then hand mixed to break up any clods which might have formed and the mixture was given a final machine blending for 1 minute. Soil-lime mixtures were placed in sealed containers to prevent moisture loss and allowed to "mellow" for an hour before molding specimens.

c. Molding of specimens.

Two-inch diameter by 4-inch (±1/8) high specimens of soil were molded at optimum moisture content using the Vicksburg miniature compactor. The mixtures were compacted so that each specimen had a dry unit weight within ± 2 pcf of the maximum unit weight determined in accordance with AASHO T-99-70. After extruding from the mold, the specimens were either sealed with Saranwrap and kept in sealed containers to preserve their molded moisture content or were subjected immediately to testing, depending on their use.

d. Selection of lime contents

A minimum of three lime contents was desirable in the strength and durability testing program. The estimated optimum lime content was selected based on the results of the pH and Atterberg limit tests described previously, and lime contents 2 percent higher and lower than the estimated were also used.

5. Results of Strength Tests

Specimens for strength tests were cured for 0, 7, 28 and 56 days at 73°F (±2). After the curing period, the specimens were unwrapped from their Saranwrap coating, weighed to determine whether moisture loss had occurred, and tested to failure at a strain rate of 0.05 inch per minute. Soil-lime mixtures have been evaluated after various curing times, but 28 days of curing seems to be the most widely used. In the ensuing discussions, only the 28-day strengths will be utilized. The strength data for other curing periods has been reported elsewhere in an extended study of accelerated curing of lime stabilized specimens (reference 15).

Figure 16 shows the relationship between strength and lime content developed for several of the soils. Various modes of response were noted. The trend of strengths for Tyler and Perrin AB shows that these mixtures will continue to increase in strength even after the addition of 14 percent and 8.5 percent lime, respectively. Lime contents in excess of these amounts were not investigated as they would be uneconomical in practice. For Perrin A and Altus subgrade, an increase in lime above the median lime content did not produce any significant increase in strength. The strength of the Dyess mixture declined slightly at its maximum lime content, whereas, the Perrin B and Houma mixtures showed a significant strength decrease at their maximum lime contents.

From Figure 16 the "strength-lime percentage" of each soil can be obtained (Table 4). The strength-lime percentage is herein defined as the lime content above which further increase in lime content resulted in 28-day unconfined compressive strengths that remained nearly constant or decreased. Altus subgrade mixture displayed the highest 28-day strength of 350 psi at 6 percent

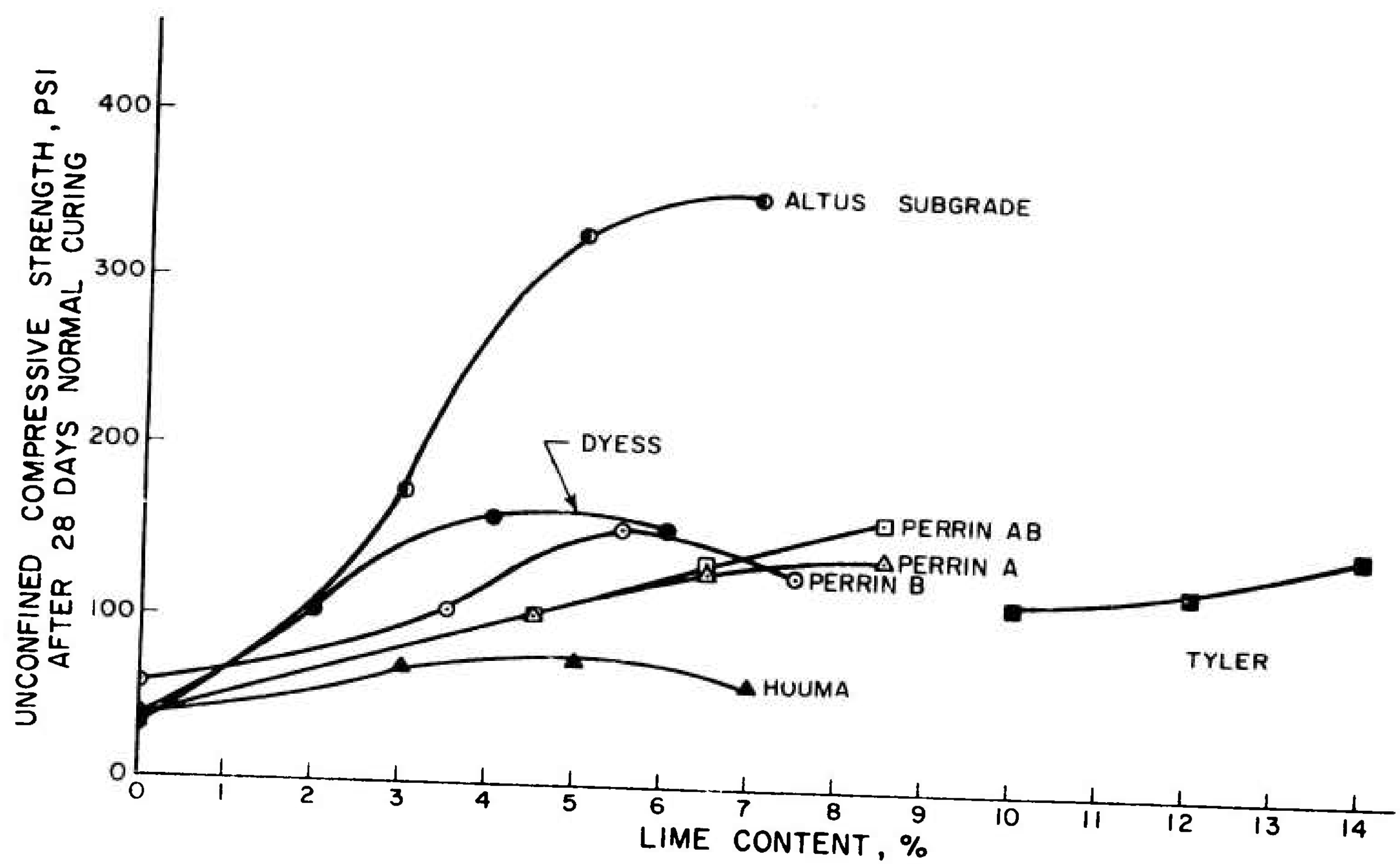


Figure 16 - Unconfined Compressive Strengths of Soil-Lime Mixtures After 28 Days Normal Curing

TABLE 4

LIME REACTIVITY[1] OF DIFFERENT SOILS

Soil	Unconfined Compressive Strength psi		Lime Reactivity Δq, psi	Strength-Lime Percentage
	Maximum 28-Day Strength obtained, q	Natural Soil Specimens		
Dyess	161.8	29.3	132.5	4.0
Altus Subgrade	356.0	36.8	319.2	6.0
Tyler	147.7	39.6	108.1	14.0[2]
Houma	77.7	36.0	41.7	5.0
Perrin A	141.1	37.3	103.8	8.5
Perrin B	156.0	58.4	97.6	5.5
Perrin AB	162.3	34.1	128.2	8.5[2]

[1]Lime reactivity is the difference between the maximum compressive strength of the soil-lime mixture and the compressive strength of the natural soil.

[2]Strength was still increasing at this lime content.

lime content, and the Houma mixture had the least strength of 77.7 psi at 5 percent lime content. The maximum strengths of Dyess, Tyler, Perrin A, Perrin B and Perrin AB were nearly the same and ranged from about 141 to 162 psi.

According to Thompson's (reference 17) definition of lime reactivity (strength increase of at least 50 psi), all soils except Houma were lime reactive (Table 4). The high organic content in the Houma soil may have prevented significant strength increase. Perrin A, Perrin B, Perrin AB and Tyler were lime reactive although they had high sulfate contents. However, previous research (reference 16) has indicated that sulfate does not have detrimental effects on the strength of lime-treated soils when the soil-lime specimens are cured at constant moisture contents not exceeding their optimum moisture contents. Thus, sulfates were not expected to prevent soils from being lime reactive.

6. Durability Tests - General

a. Applicability of laboratory durability tests.

Satisfactory strength retained (residual strength) in a soil-lime mixture after being subjected to durability tests is a meaningful measure of durability of the mixture. However, conventional laboratory durability test conditions do not exactly simulate the field conditions, and the durability properties obtained in the laboratory do not reflect those in the field. It has been shown (references 18 and 19) that soil-lime mixtures, when protected from weathering by a bituminous wearing surface, are more durable than most laboratory durability tests would indicate. Thus, it would appear that laboratory durability tests have as their primary use the determination of relative durabilities of different soil-lime mixtures until that point in time when extensive correlation between laboratory and field durability has been accomplished.

b. Test procedures.

Unconfined compression specimens were molded, using the procedure described earlier, at the three lime contents utilized for the strength testing. All specimens were sealed in Saranwrap and placed in sealed containers for curing. Three specimens were cured for 7 days at 73°F (±2) after which the Saranwrap was removed and the specimens were completely immersed in water for the next 21 days. This was termed the <u>long-term immersion</u> test. The remaining specimens were cured for 28 days at 73°F (±2). Three of these specimens were immersed in water at 73°F (±2) for 24 hours. This was the <u>short-term immersion</u> test.

Unconfined compression tests were performed on the soaked specimens and on unsoaked specimens which were molded at the same time and had undergone curing at the same temperatures.

Additional specimens, after being cured for 28 days at 73°F (±2) were subjected to freeze-thaw cycles. The freeze-thaw technique was that given in ASTM Designation: D560-67, with the exception that 2-inch diameter by 4-inch high specimens were used instead of 4-inch diameter by 4.6-inch high (Proctor) specimens, and the test consisted of 10 cycles of 16 hours freezing and 8 hours thawing. In addition to compressive strength determinations, measurements of unit length and moisture change were made after various cycles of freezing and thawing.

7. <u>Short-Term Immersion Test</u>

The general trend of strengths after short-term immersion was similar for all lime-treated soils, showing a loss in strength with respect to the 28-day strengths (Figures 17 through 23). Figure 24(a) compares the short-term immersed strengths of all soil-lime mixtures. It was observed that for

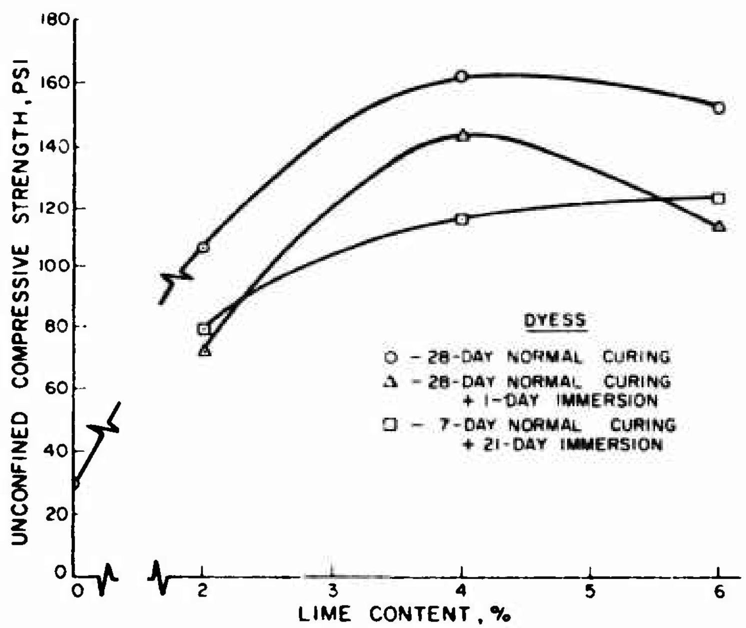


Figure 17 - Initial and Immersed Unconfined Compressive Strengths of Lime-Treated Dyess Soil

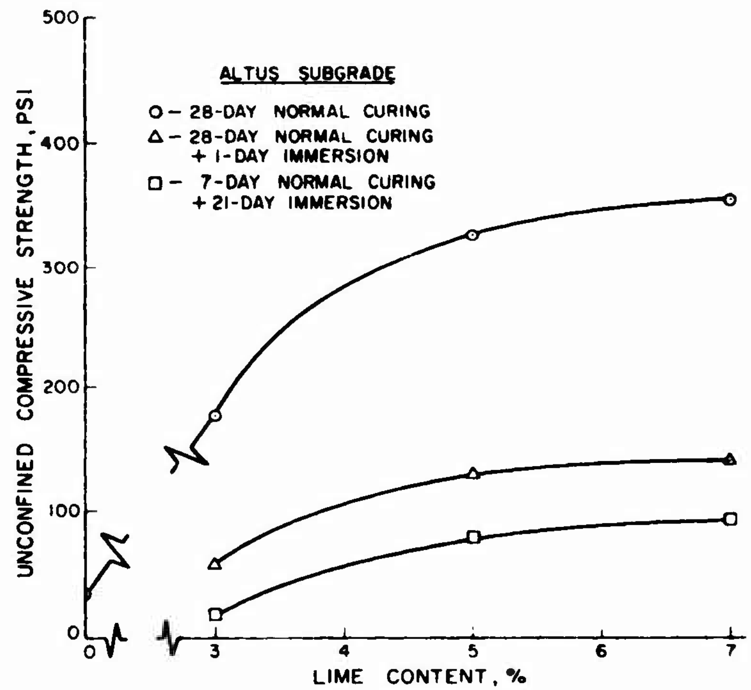


Figure 18 - Initial and Immersed Unconfined Compressive Strengths of Lime-Treated Altus Subgrade Soil

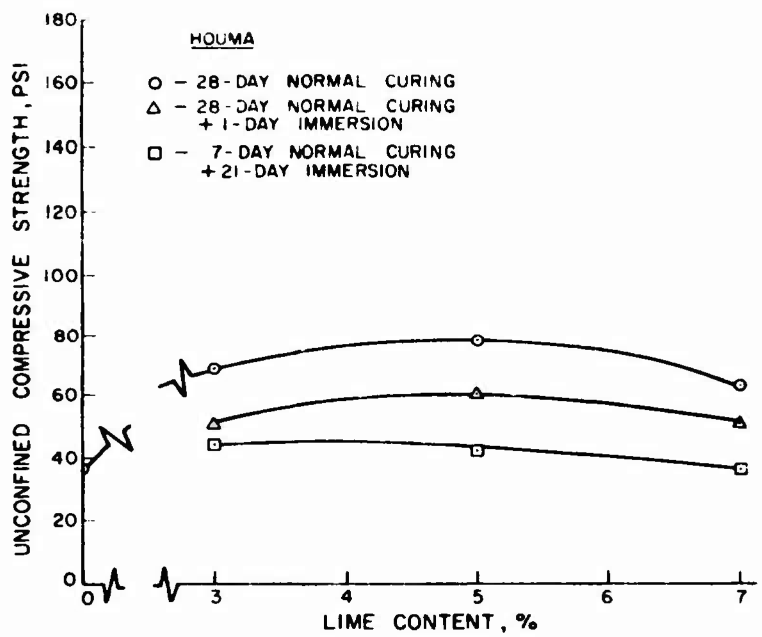


Figure 19 - Initial and Immersed Unconfined Compressive Strengths of Lime-Treated Houma Soil

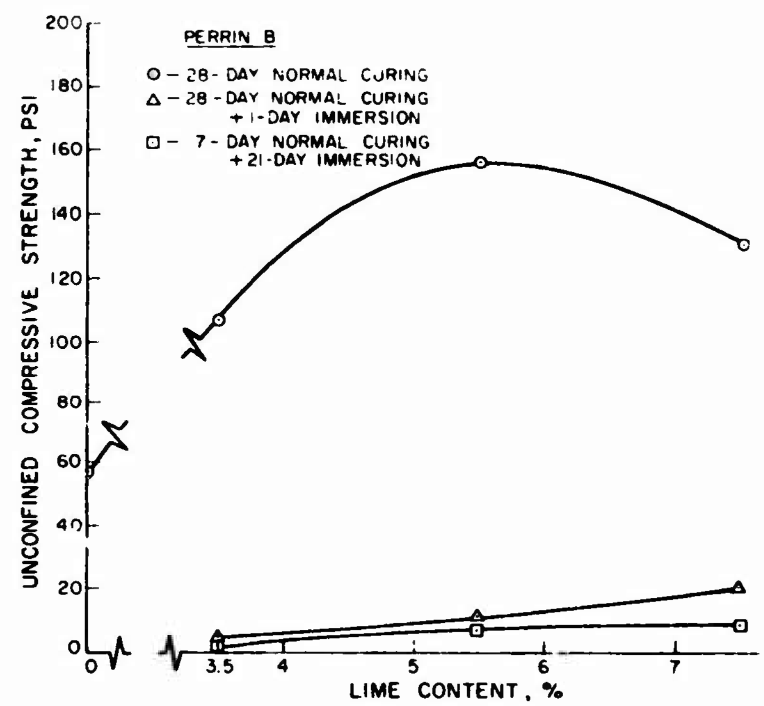


Figure 20 - Initial and Immersed Unconfined Compressive Strengths of Lime-Treated Perrin B Soil

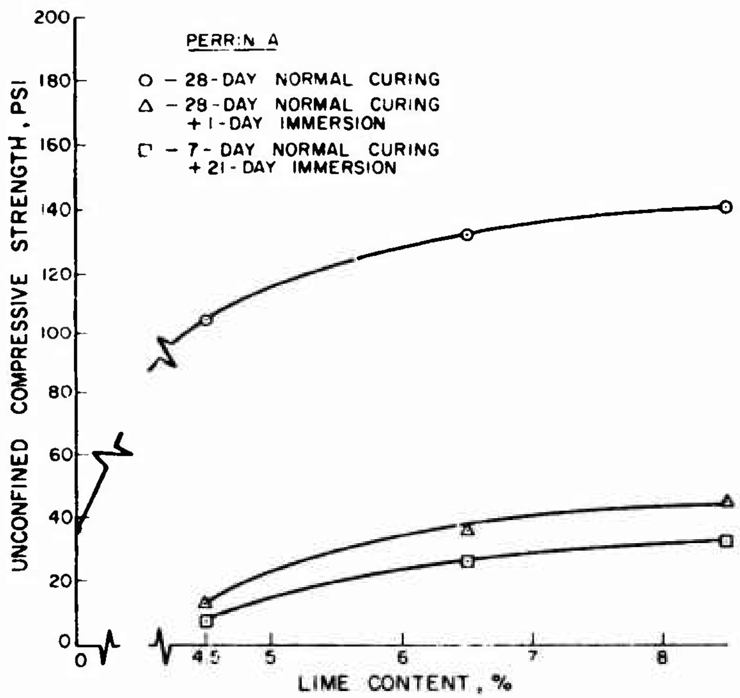


Figure 21 - Initial and Immersed Unconfined Compressive Strengths of Lime-Treated Perrin A Soil

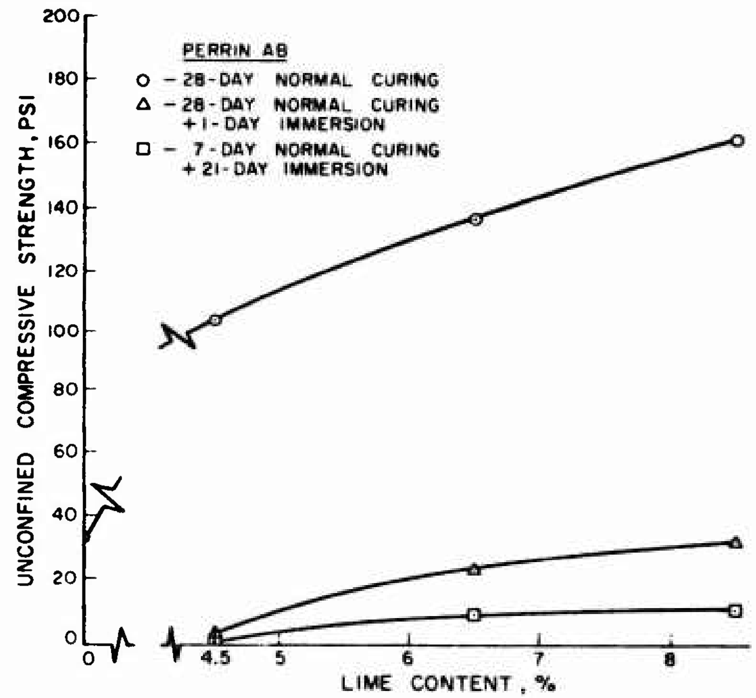


Figure 22 - Initial and Immersed Unconfined Compressive Strengths of Lime-Treated Perrin AB Soil

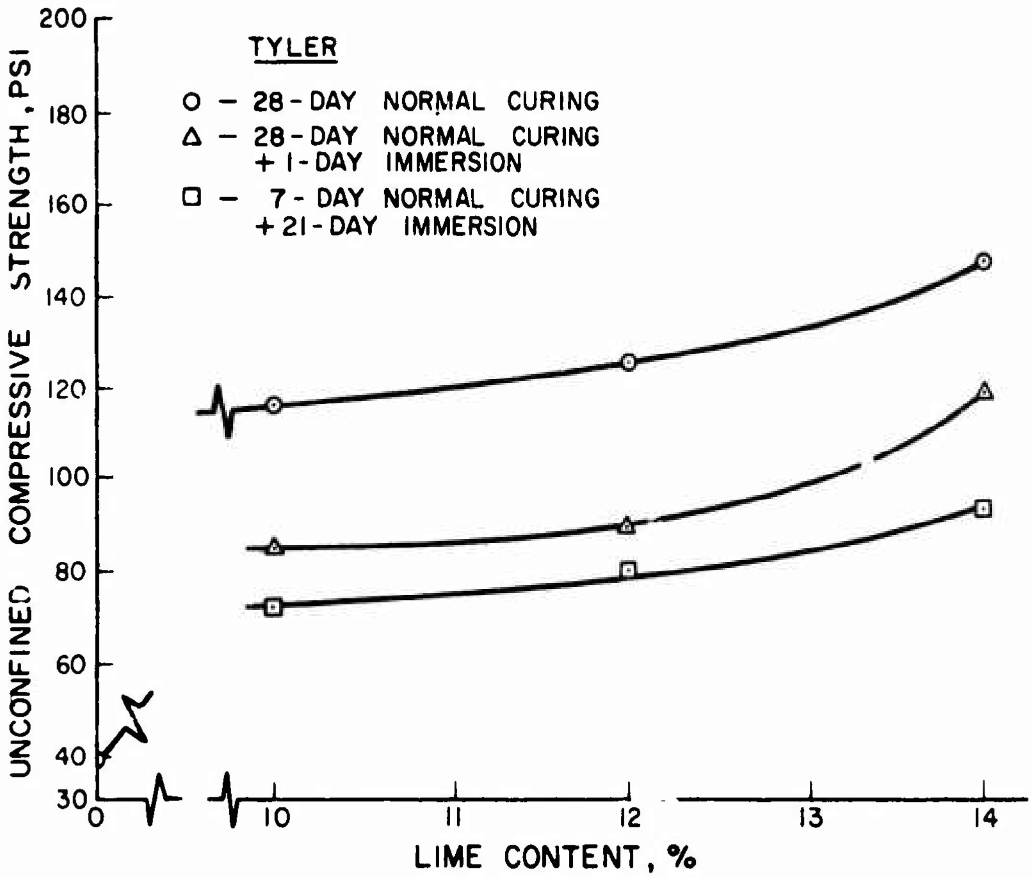


Figure 23 - Initial and Immersed Unconfined Compressive Strengths of Lime-Treated Tyler Soil

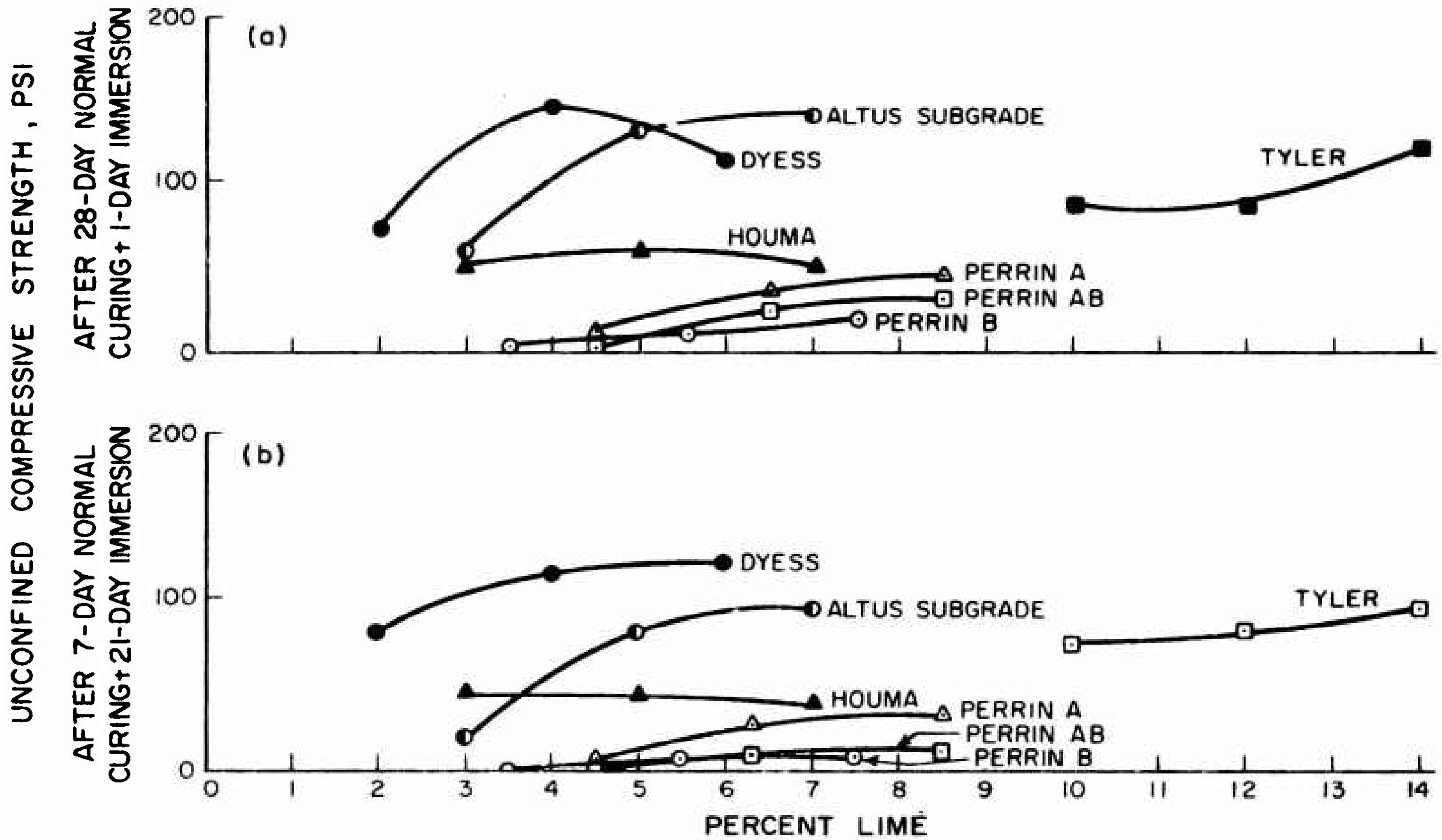


Figure 24 - Immersed Unconfined Compressive Strengths of Different Soil-Lime Mixtures With Varying Amount of Lime

all soils except Tyler, the minimum lime contents giving the highest short-term immersed strengths were nearly the same as the strength-lime percentages of the soils. For the Tyler soil, it appears that the lime content giving maximum immersed strength would be well above 14 percent.

A comparison of short-term immersed strengths of different soils treated with their respective strength-lime percentages is shown in Figure 25. Dyess, Altus subgrade, and Tyler exhibited the highest unconfined compressive strengths after short-term immersion, ranging from about 119 to 144 psi, and thus appeared to be quite durable. The Altus subgrade-mixture 28-day strength was more than twice that of the Dyess or Tyler mixtures, but all three had approximately the same immersed strength; or put another way, the Altus subgrade mixture lost more strength during immersion than the Dyess or Tyler mixtures. This can be expressed in terms of an index of resistance to short-term immersion, R_i, defined as the strength after immersion divided by the original strength. As shown in Table 5, the Altus subgrade mixture had less resistance (R_i = 39.3 percent) to short-term immersion than Dyess and Tyler mixtures even though it possessed almost the same immersion durability as the other two mixtures did. The Houma mixture had a reasonably high index of resistance to short-term immersion, but its low immersed strength (60 psi) indicates that it was not durable. Perrin B, Perrin A and Perrin AB were not durable in short-term immersion.

The soil-lime mixtures of Dyess, Tyler, Perrin A, Perrin B and Perrin AB had nearly the same 28-day strengths, but their short-term immersed strengths varied widely (Figure 25). This would indicate that there is no direct correlation between the initial 28-day strength and short-term immersion durability of these soil-lime mixtures.

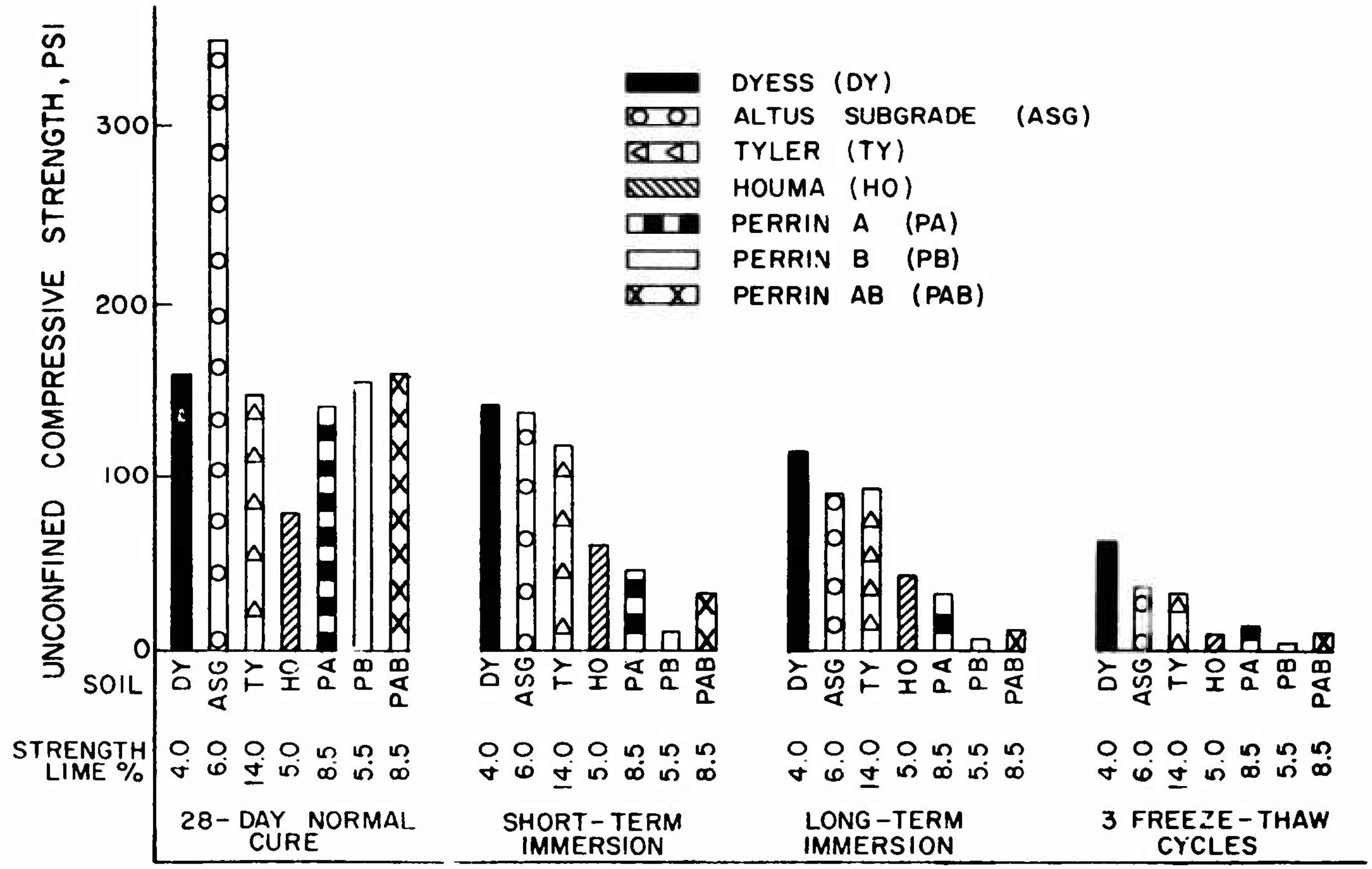


Figure 25 - Comparison of Unconfined Compressive Strengths of Soils Treated With Strength-Lime Percentages

8. Long-Term Immersion Test

The lime-treated soil specimens were subjected to a 7-day normal curing and 21-day immersion in water to measure their long-term immersion durability. Figures 17 through 23 show a general decrease in strength after immersion compared to the 28-day strength, as might be expected. Figure 24(b) shows long-term immersed strengths of all the soils at varying lime contents. It was noted that a lime content in excess of the strength-lime percentage did not have a beneficial effect on the long-term immersed strength of any of the soil-lime mixtures. For two soils, Houma and Perrin B, it had an adverse effect.

Dyess and Tyler mixtures displayed substantial resistance to long-term immersion as measured by their indices of resistance to long-term immersion (R_{ii}) of 73.0 and 63.6 percent respectively (Table 5). The Altus subgrade mixture had less resistance to long-term immersion (R_{ii} = 25.7 percent) than Dyess and Tyler (Table 5), indicating that the mixture lost its initial strength at a faster rate during long-term immersion. While the Houma soil did not lose significant strength during long-term immersion (R_{ii} was high), its strength was still too low to be classed as durable. Low immersed strengths and low indices of resistance to long-term immersion observed for Perrin A, Perrin B and Perrin AB mixtures indicated that these mixtures did not possess immersion durability.

The Tyler mixture, even though it had high organic and sulfate content, possessed good immersion durability. Research by Lambe et al. (reference 20) showed that sodium sulfate was uniquely effective in cement stabilization of sandy soils containing organic matter. Possibly, the combination of sulfate and organics in the Tyler soil had the same effect noted by Lambe as long as the mixture was cured at normal temperature (73°F). However, it was observed that when the chemical reaction was accelerated by curing the Tyler mixture at

TABLE 5

UNCONFINED COMPRESSIVE STRENGTHS AND INDICES OF RESISTANCE

TO DIFFERENT DURABILITY TESTS OF SOIL-LIME MIXTURES

AT THEIR STRENGTH-LIME CONTENTS

Soil	Strength-Lime %[a]	Maximum 28-Day Strength psi	Short-Term Immersed Strength, psi	Long-Term Immersed Strength, psi	Unconfined Compressive Strength After 3 F-T Cycles psi	R_i[b] %	R_{ii}[c] %	R_f[d] %
Dyess	4.0	161.8	144.0	118.0	63.0	89.0	73.0	39.0
Altus Subgrade	6.0	350.0	137.5	90.0	35.0	39.3	25.7	10.0
Tyler	14.0	147.7	119.0	94.0	33.0	80.6	63.6	22 4
Houma	5.0	77.7	60.0	44.0	8.0	77.4	56.6	10.3
Perrin A	8.5	141.1	45.0	32.0	13.0	31.9	22.7	9.2
Perrin B	5.5	156.0	11.0	8.0	4.5	7.1	7.1	2.9
Perrin AB	8.5	162.3	32.0	11.0	10.0	19.7	6.8	6.2

[a] Lime percentages used for Tyler and Perrin AB are maximum lime percentages. Strength-lime percentages for those soils were not determined.

[b,c,d] As defined in Appendix L.

an elevated temperature for a short period (reference 15), its strength considerably decreased. It is, therefore, thought that if the mixture were allowed to cure a considerable time at, say, 73°F in the presence of water, a slow reaction process between organic matter, sulfate, and lime might occur, resulting in progressive deterioration in strength of the mixture.

9. Freeze-Thaw Test

a. Unconfined compressive strength.

Residual strengths after freeze-thaw cycles were used as a measure of freeze-thaw (F-T) durability of lime-stabilized soils. However, the test procedure was so severe that the strengths after 4 freeze-thaw cycles were not significant enough to use in evaluating the freeze-thaw durability of different soil-lime mixtures.

Figure 26 shows unconfined compressive strengths after 3 freeze-thaw cycles of the different soils treated with varying amounts of lime. Results for all cycles are shown in Figures 27 through 33. It is noticed from these figures that unconfined compressive strengths of the specimens decreased sharply after 2 to 3 cycles of freezing and thawing. With additional freeze-thaw cycles, the strengths declined slowly until there was little or no strength left after 10 cycles. Figures 34 through 40 pictorially show the deterioration of the specimens after 10 freeze-thaw cycles.

With the exception of the Tyler mixture, freeze-thaw durability (as measured by unconfined compressive strength after 3 freeze-thaw cycles) of all soil-lime mixtures did not significantly increase for lime contents above their strength-lime percentages (Figure 26). Figure 25 compares the durabilities after 3 freeze-thaw cycles of different soils treated with their respective strength-lime percentages. The Dyess mixture exhibited the maximum

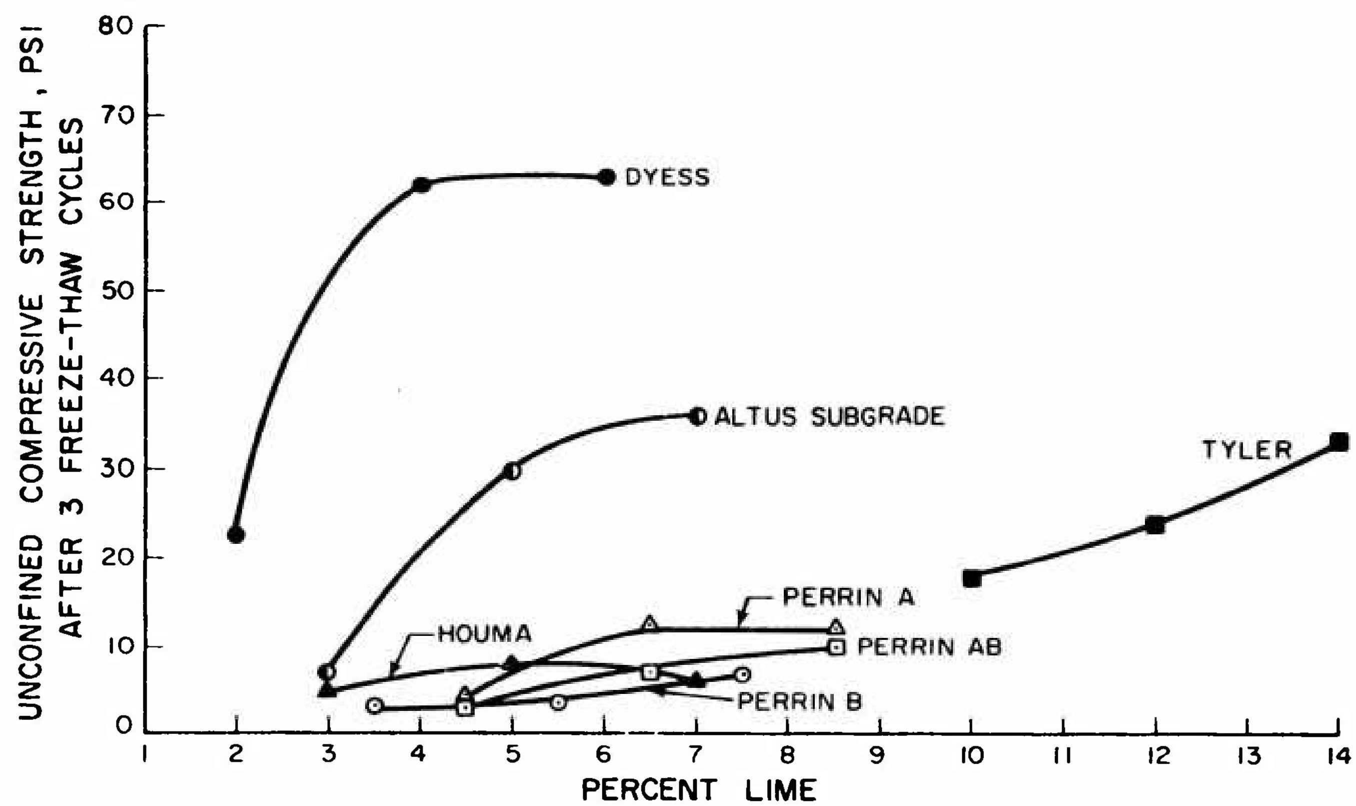


Figure 26 - Unconfined Compressive Strength After 3 Freeze-Thaw Cycles of Different Soil-Lime Mixtures With Varying Amount of Lime

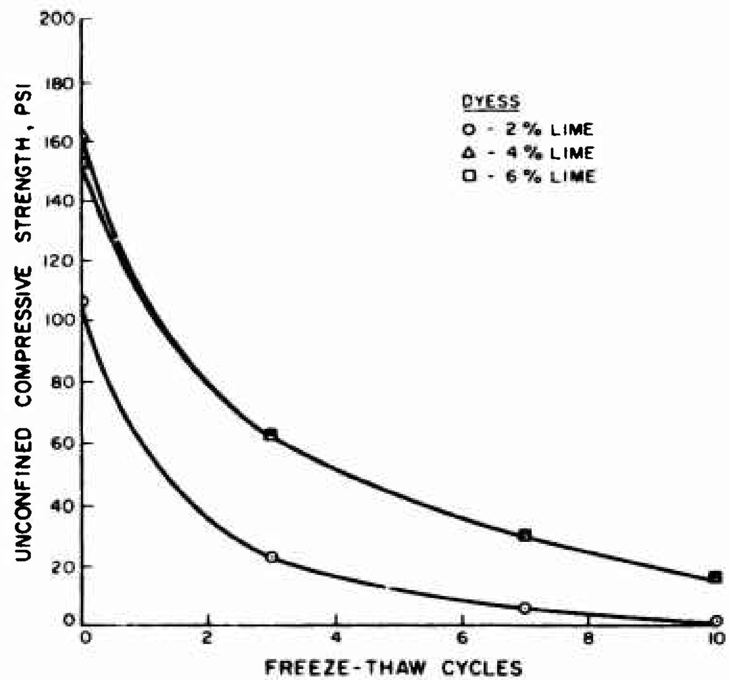


Figure 27 - Influence of Freeze-Thaw Cycles on Unconfined Compressive Strength of Dyess Soil

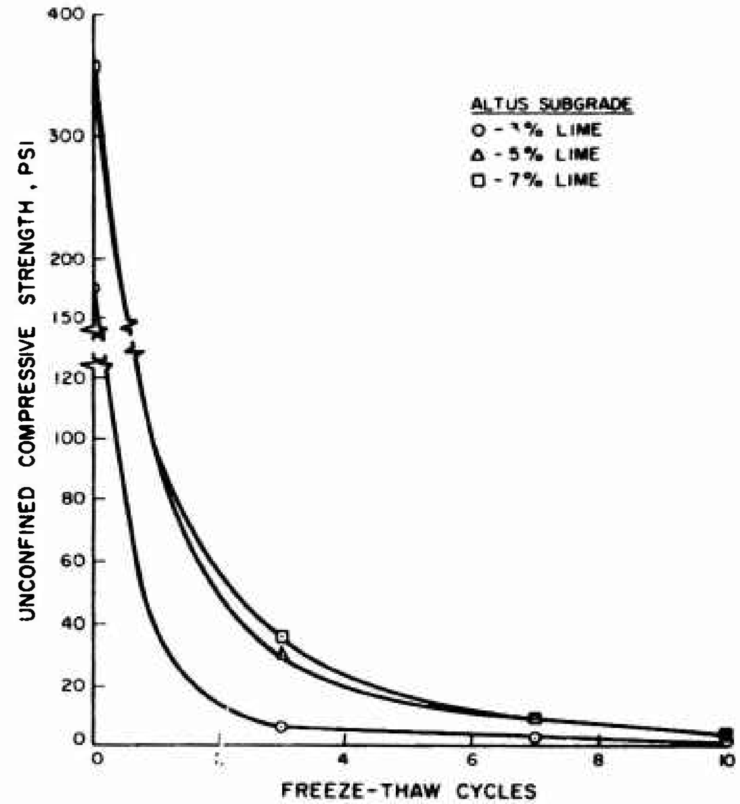


Figure 28 - Influence of Freeze-Thaw Cycles on Unconfined Compressive Strength of Altus Subgrade Soil

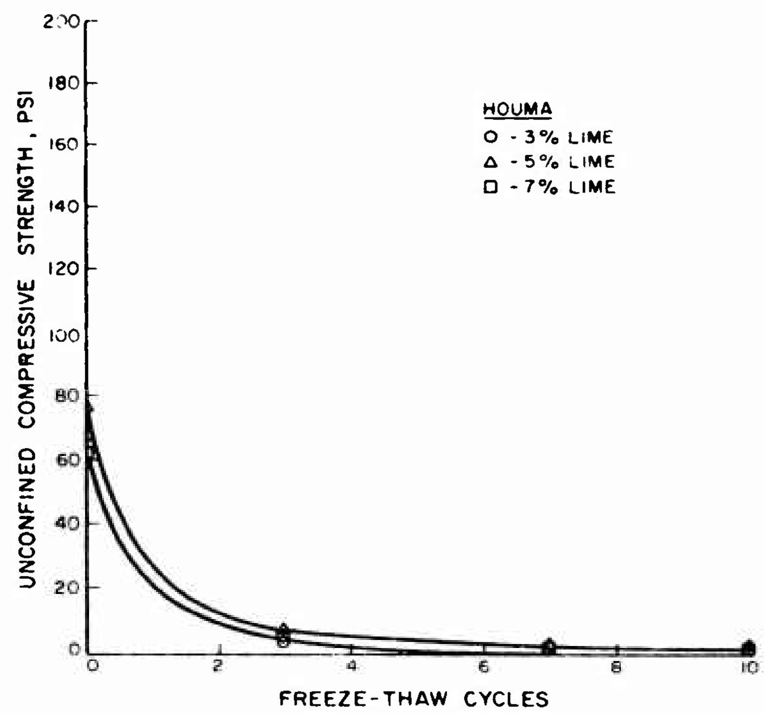


Figure 29 - Influence of Freeze-Thaw Cycles on Unconfined Compressive Strength of Houma Soil

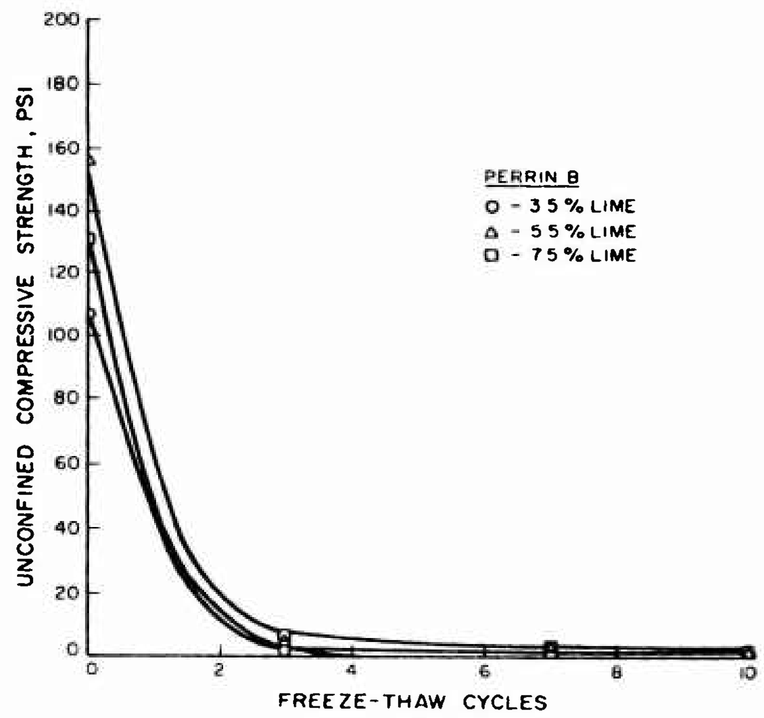


Figure 30 - Influence of Freeze-Thaw Cycles on Unconfined Compressive Strength of Perrin B Soil

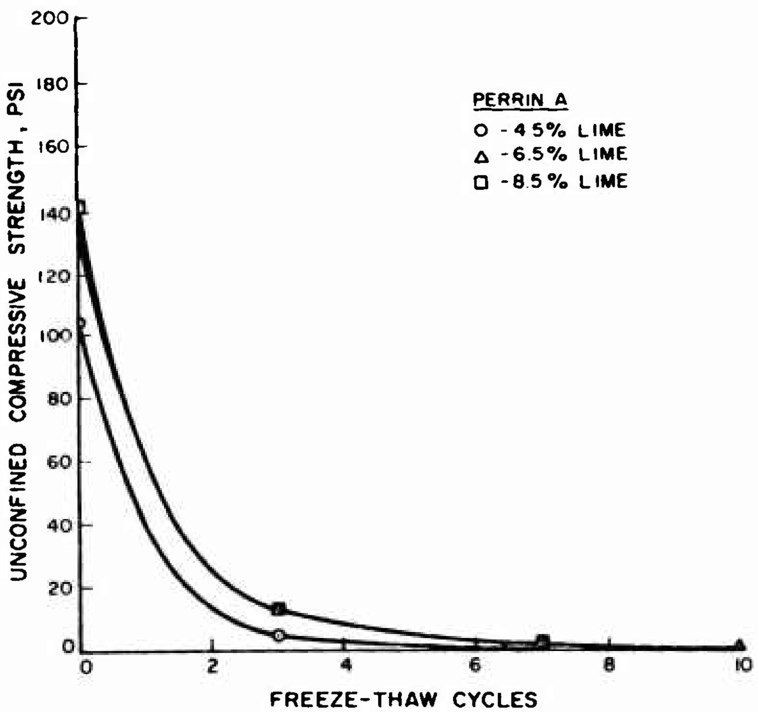


Figure 31 - Influence of Freeze-Thaw Cycles on Unconfined Compressive Strength of Perrin A Soil

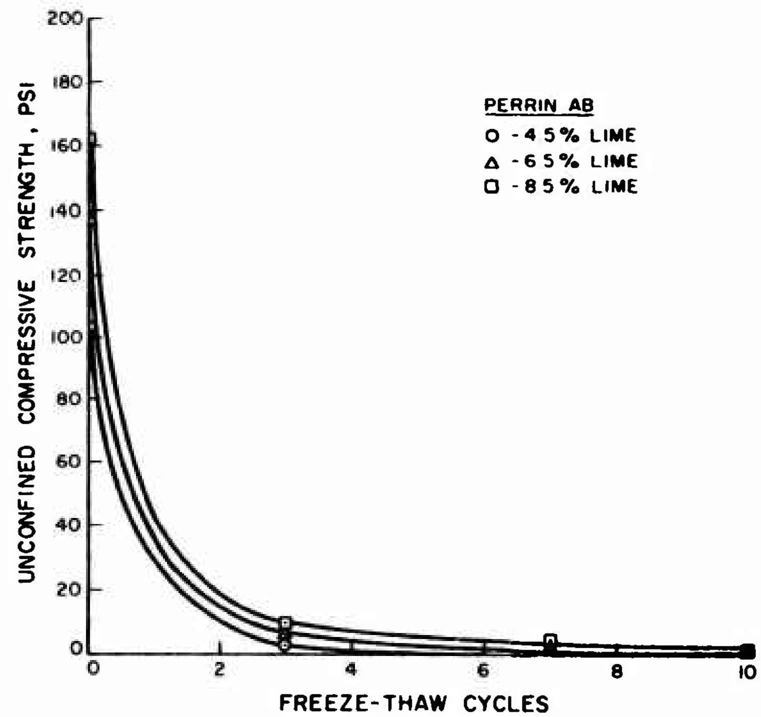


Figure 32 - Influence of Freeze-Thaw Cycles on Unconfined Compressive Strength of Perrin AB Soil

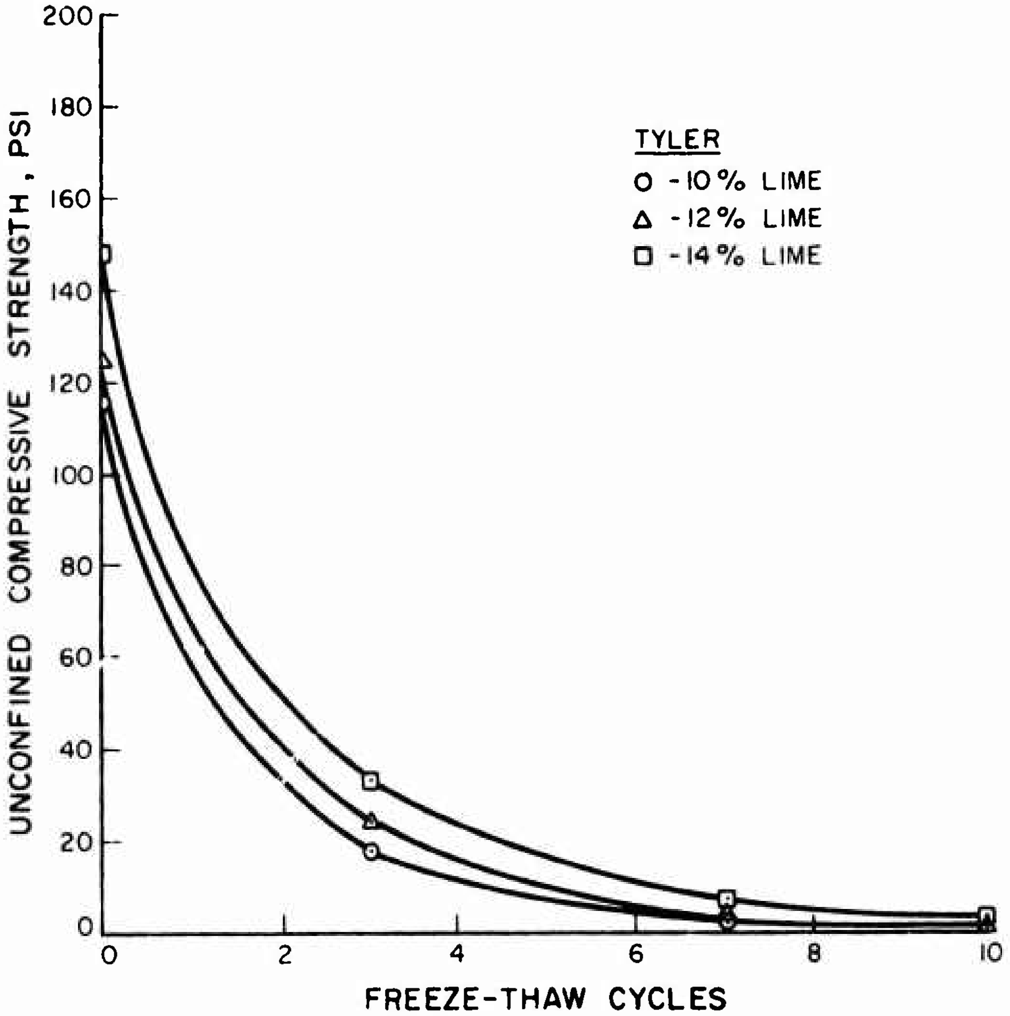


Figure 33 - Influence of Freeze-Thaw Cycles on Unconfined Compressive Strength of Tyler Soil

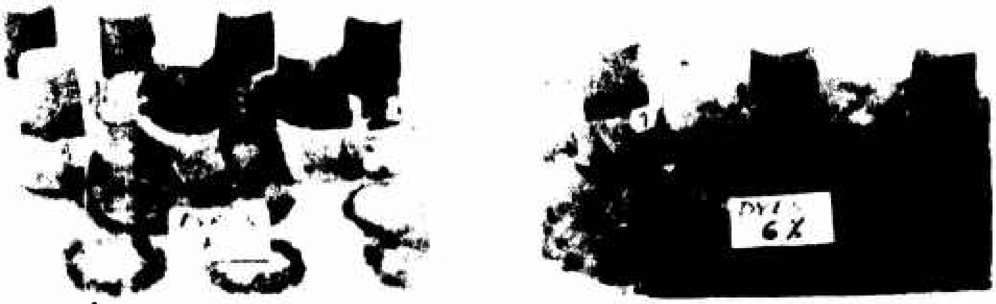


Figure 34 - Lime-Treated Dyess Specimens Subjected to 10 Freeze-Thaw Cycles. Numbers Indicate Percent Lime

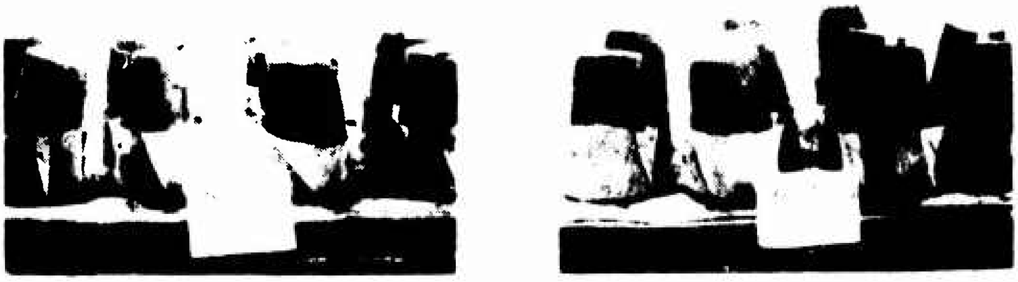


Figure 35 - Lime-Treated Altus Subgrade Specimens Subjected to 10 Freeze-Thaw Cycles. Numbers Indicate Percent Lime

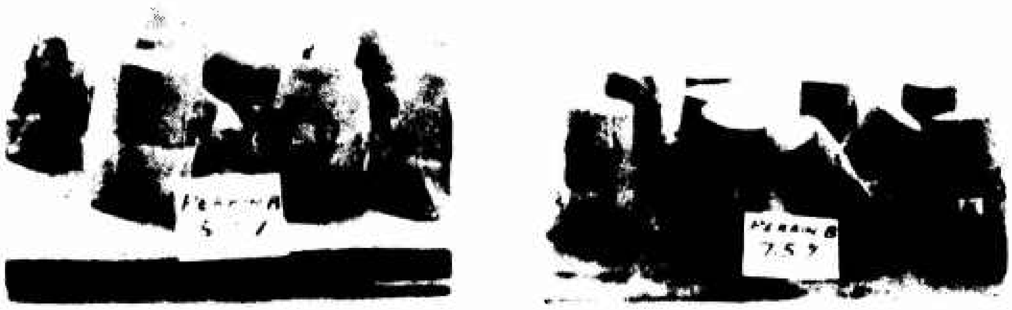


Figure 36 - Lime-Treated Perrin B Specimens Subjected to 10 Freeze-Thaw Cycles. Numbers Indicate Percent Lime

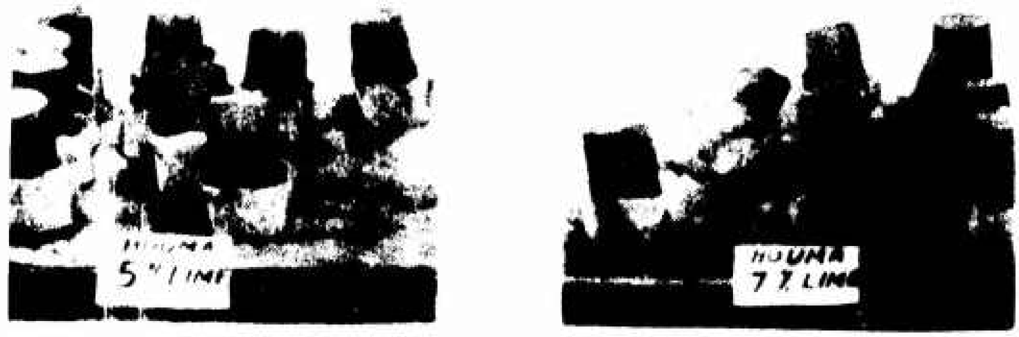


Figure 37 - Lime-Treated Houma Specimens Subjected to 10 Freeze-Thaw Cycles. Numbers Indicate Percent Lime

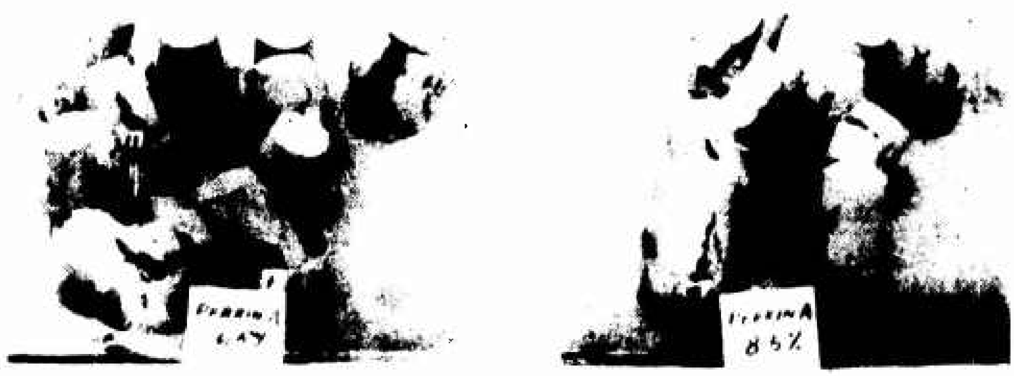


Figure 38 - Lime-Treated Perrin A Specimens Subjected to 10 Freeze-Thaw Cycles. Numbers Indicate Percent Lime

Figure 39 - Lime-Treated Perrin AB Specimens Subjected to 10 Freeze-Thaw Cycles. Numbers Indicate Percent Lime

Figure 40 - Lime-Treated Tyler Specimens Subjected to 10 Freeze-Thaw Cycles. Numbers Indicate Percent Lime

residual strength; it also showed the maximum index of resistance (39.0 percent) to freezing and thawing (Table 5), and appeared to be the most durable against freeze-thaw action. The initial 28-day strength of the Altus subgrade mixture was much greater than that of the Tyler mixture, but the lower index of resistance to freezing and thawing observed for the Altus subgrade indicated that its strength decreased at a faster rate with freeze-thaw cycles. However, Altus subgrade and Tyler mixtures retained almost the same residual strength (35 and 33 psi, respectively) after 3 cycles of freezing and thawing, and both soils appeared to possess reasonable freeze-thaw durability. Houma, Perrin A, Perrin B and Perrin AB mixtures had little strength after 3 cycles of freezing and thawing, indicating that they were not durable.

b. Unit length change.

Plots for unit length change with respect to freezing and thawing cycles are shown in Figures 41 through 44. After 7 freeze-thaw cycles all specimens slumped under their own weights producing unreliable measurements. Dyess, Houma and Tyler mixtures displayed smaller unit length changes (less than 0.05 in/in) after 7 freeze-thaw cycles than the other soil-lime mixtures (Figures 41, 42 and 43). It appears that increasing the lime percentage in any of these soil-lime mixtures did not significantly influence the unit length change. The Altus subgrade mixture, which showed high residual strength after 3 cycles of freezing and thawing, experienced relatively large unit length changes (Figure 41). Perrin B, Perrin A and Perrin AB experienced relatively large unit length changes after 7 cycles of freezing and thawing. These 3 mixtures also displayed very low residual strength after freeze-thaw cycles.

The test results show that unit length change was not a satisfactory measure of freeze-thaw durability of soil-lime mixtures. For example, the Houma mixture, which was not classed as durable, had higher resistance to unit length changes than the more durable Altus subgrade mixture.

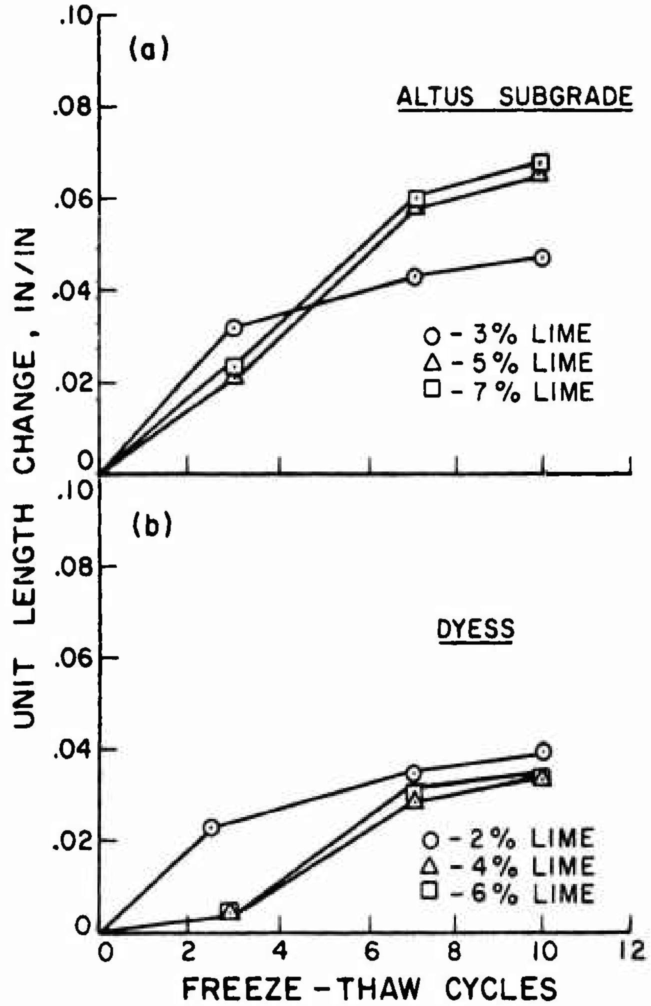


Figure 41 - Influence of Freeze-Thaw Cycles on Unit Length Changes of Lime-Treated Altus Subgrade and Dyess Soils

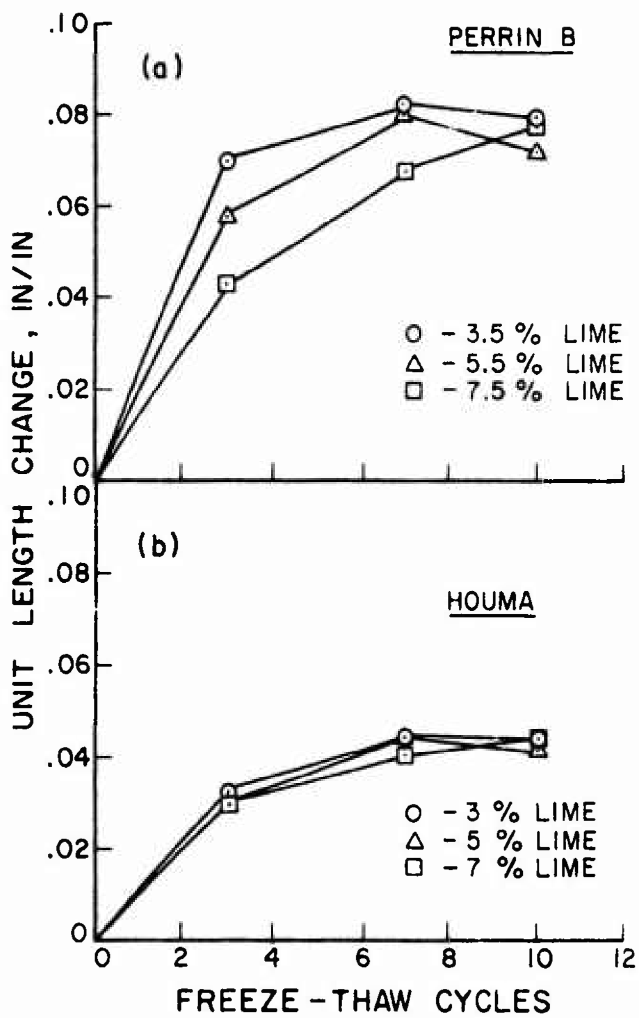


Figure 42 - Influence of Freeze-Thaw Cycles on Unit Length Changes of Lime-Treated Perrin B and Houma Soils

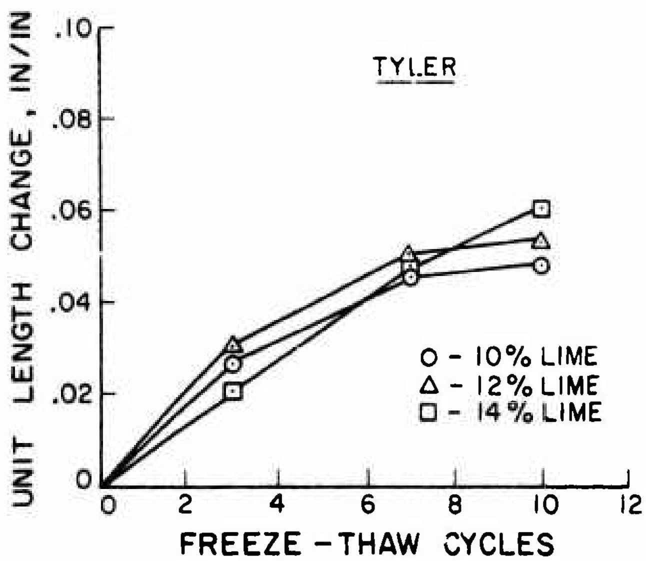


Figure 43 - Influence of Freeze-Thaw Cycles on Unit Length Change of Lime-Treated Tyler Soil

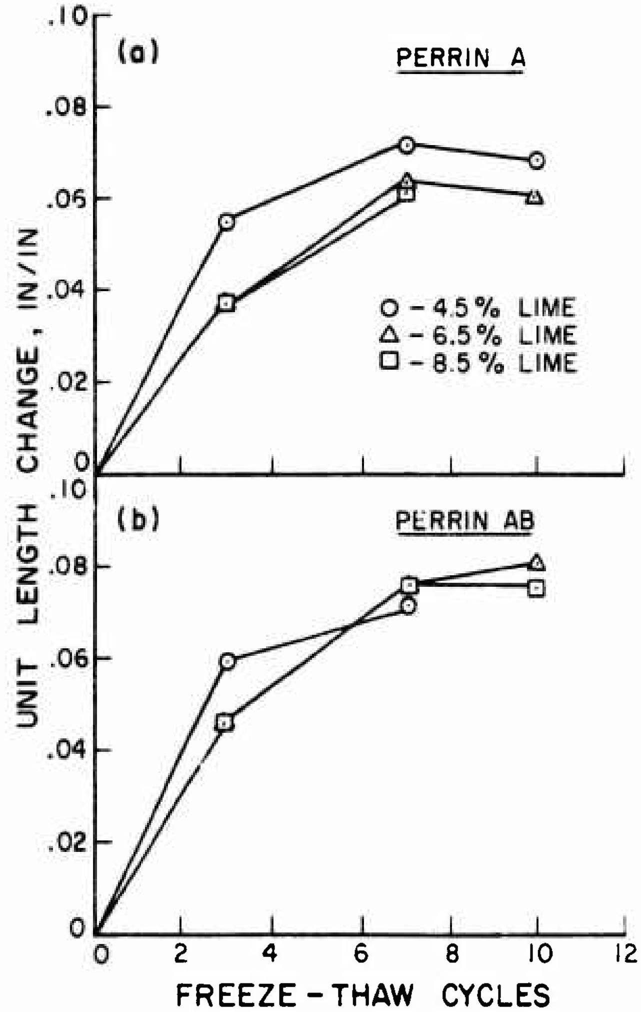


Figure 44 - Influence of Freeze-Thaw Cycles on Unit Length Changes of Lime-Treated Perrin A and Perrin AB Soils

c. Moisture content.

Plots of moisture content after various cycles of freezing and thawing (Figures 45 through 48) show that the moisture contents of all soil-lime mixtures increased as the number of freeze-thaw cycles increased. Except for Dyess, all soil-lime specimens molded at their median lime contents exhibited rapid moisture content increase. Dyess, which possessed good freeze-thaw durability properties, exhibited a slow rate of moisture increase with freeze-thaw cycles (Figure 45). Houma specimens, which could not be classed as durable, showed moisture content increases of about 18 percent indicating that the mixture had reasonably high resistance to moisture changes as compared to the remaining soils. Altus subgrade and Tyler mixtures were durable mixtures as measured by their residual strength after 3 cycles of freezing and thawing, but they did not have substantial resistance to moisture changes. The two mixtures molded at their median lime contents had moisture increases of about 24 percent and 29 percent respectively after 10 cycles of freezing and thawing. Rapid and large moisture content increases observed in Perrin B, Perrin A and Perrin AB specimens during freezing and thawing indicated that they were not durable. These mixtures also displayed low unconfined compressive strengths and greater unit length changes after freeze-thaw cycles.

10. Relationship Between Durability Tests

As mentioned previously, the three durability tests represent efforts to predict relative durability between lime-stabilized soils, rather than an attempt to predict actual durability in a pavement structure. However, the freeze-thaw durability test may be more applicable in northern climates whereas the immersion tests would be more suitable for warm, humid climates. The

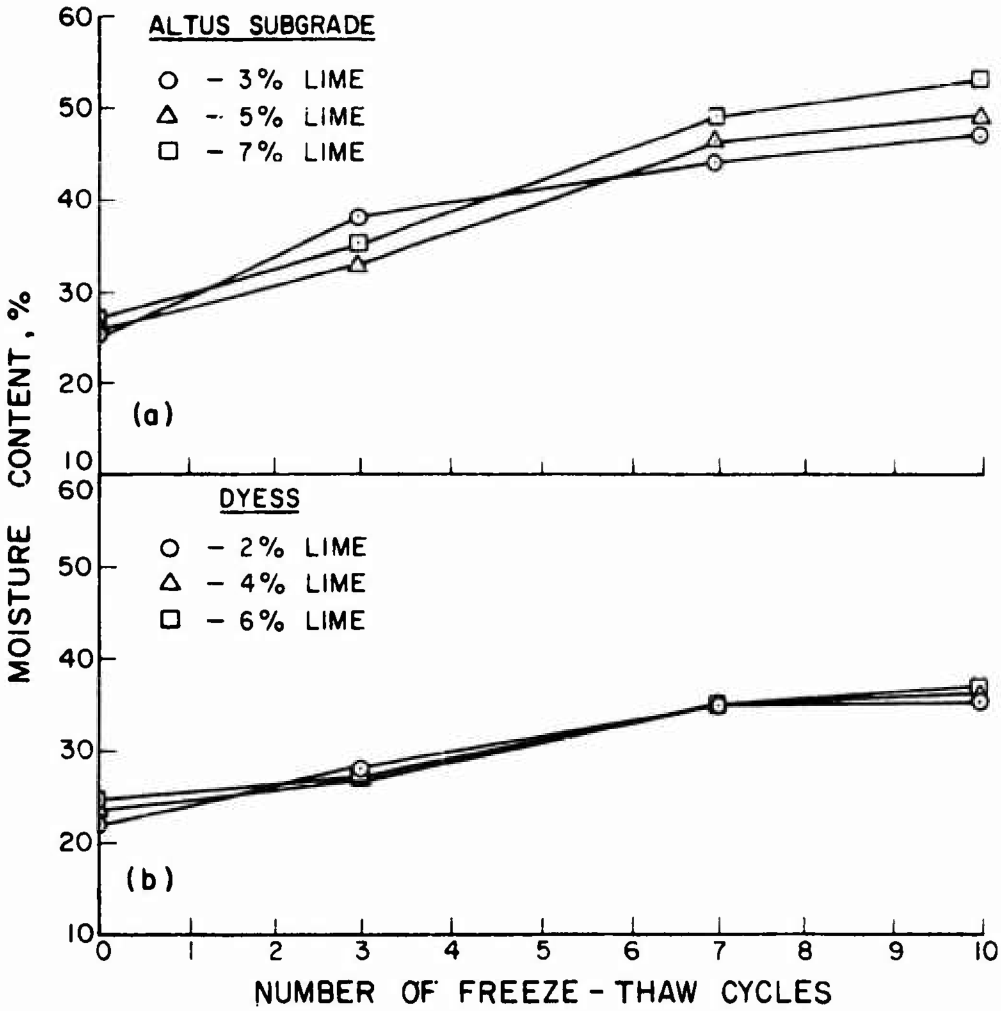


Figure 45 - Moisture Contents of Lime-Treated Altus Subgrade and Dyess Soils After Freeze-Thaw Cycles

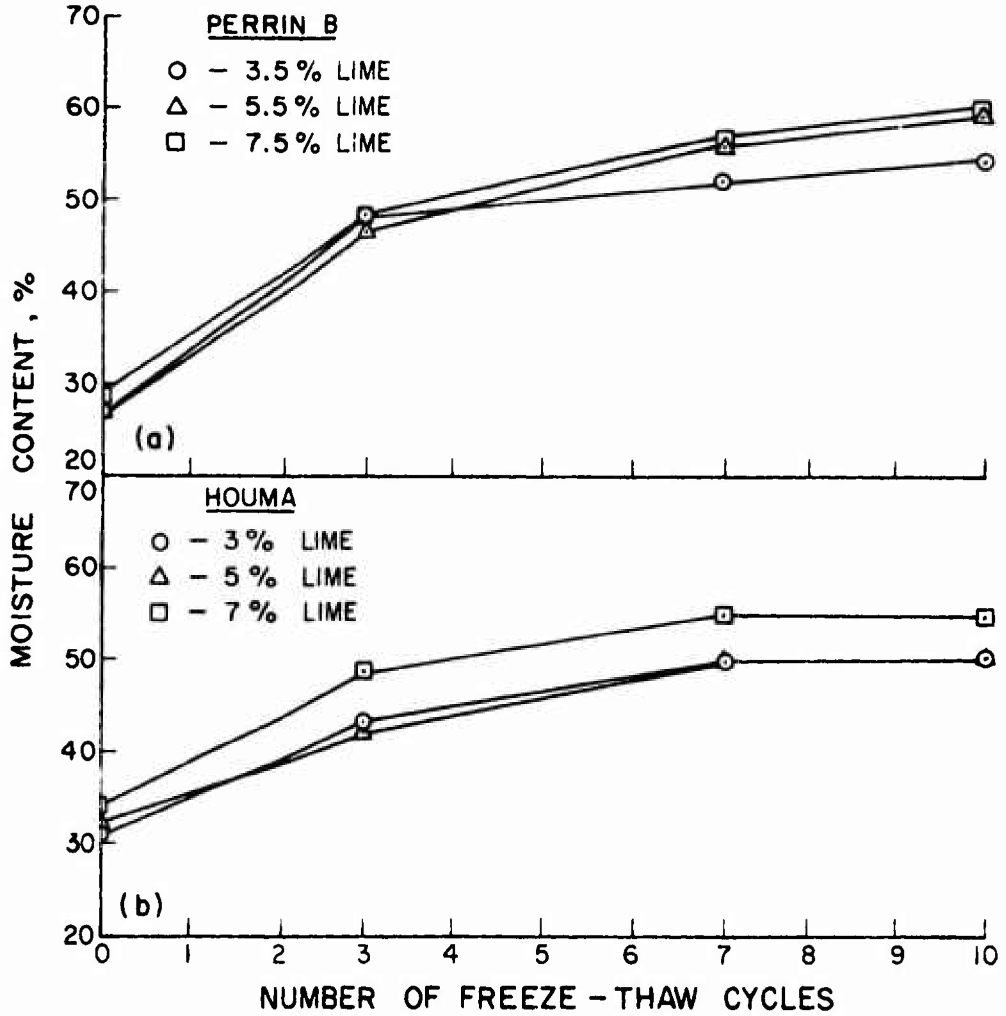


Figure 46 - Moisture Contents of Lime-Treated Perrin B and Houma Soils After Freeze-Thaw Cycles

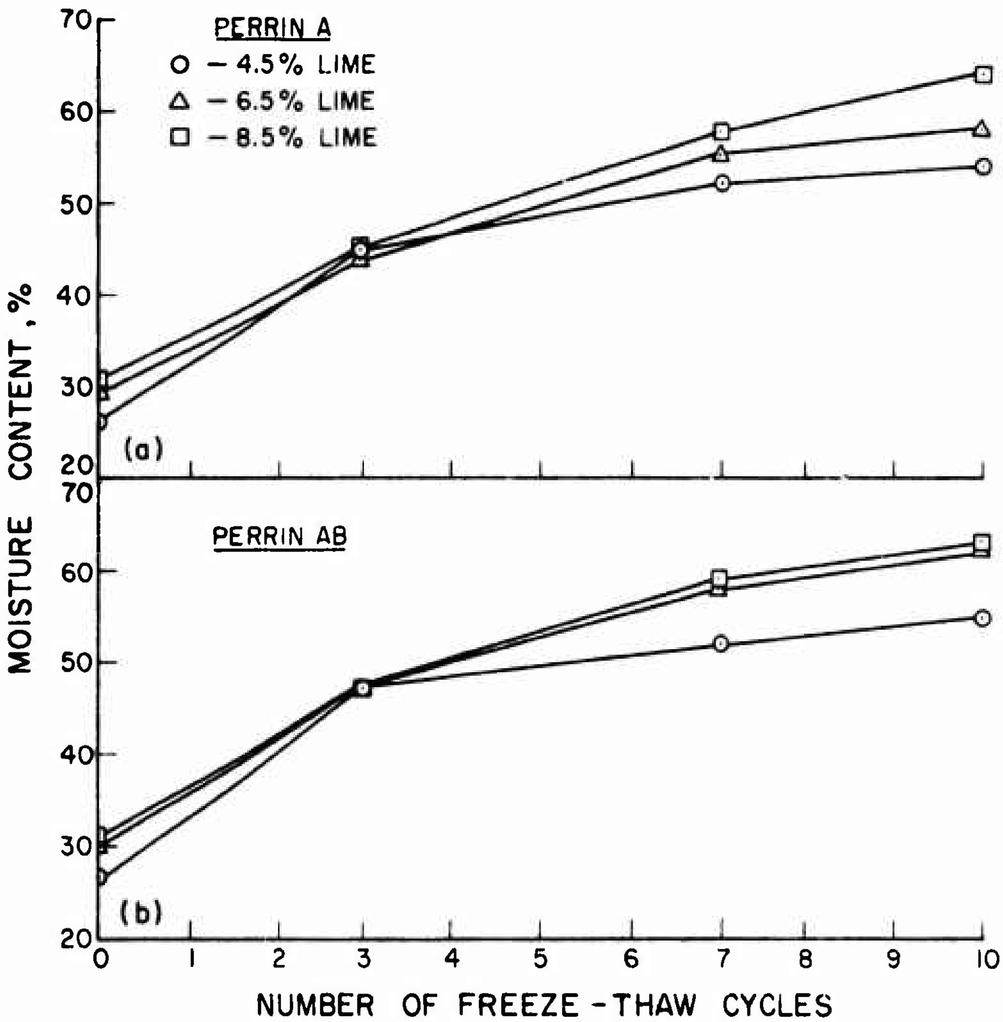


Figure 47 - Moisture Contents of Lime-Treated Perrin A and Perrin AB Soils After Freeze-Thaw Cycles

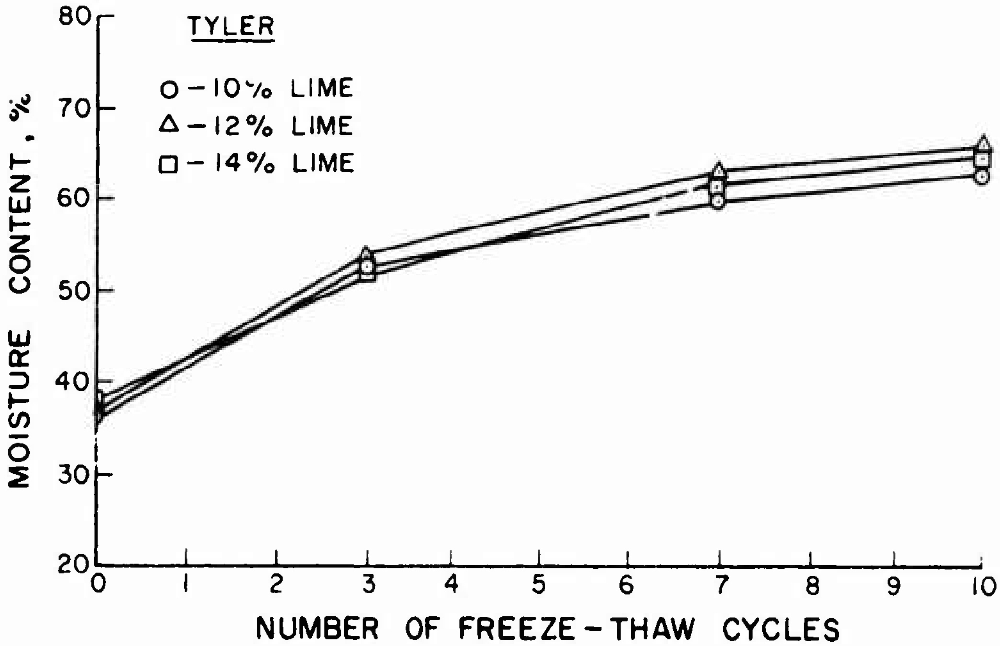


Figure 48 - Moisture Content of Lime-Treated Tyler Soil After Freeze-Thaw Cycles

freeze-thaw test requires more equipment (a freezing chest or chamber, at the minimum) and more time to perform than the immersion tests. With this in mind, it was worthwhile to see if a correlation existed between the various durability tests so that one test could be substituted for the other.

a. Relation between freeze-thaw and immersion.

Semilog plots of unconfined compressive strengths after 3 freeze-thaw cycles versus short-term or long-term immersed strengths showed a linear relationship (Figures 49 and 50). As might be expected, these figures reveal that high immersed strength was indicative of good freeze-thaw durability. A statistical analysis of the data revealed the following relationships:

(1) For freeze-thaw and short-term immersion

$$\log_{10} q_f = \log_{10} 3.438 + 0.00846\, q_i$$

Correlation coefficient, R = 0.91

(2) For freeze-thaw and long-term immersion

$$\log_{10} q_f = \log_{10} 3.994 + 0.00982\, q_{ii}$$

Correlation coefficient, R = 0.93

where: q_f = strength after short-term immersion

q_i = strength after long-term immersion

q_{ii} = strength after long-term immersion

Since these correlations are based on a limited number of soils, they should not be generalized for all soils.

b. Relation between long-term and short-term immersion.

A linear relationship existed between the long-term and short-term immersed strengths (Figure 51). Statistical analysis of the data revealed the following:

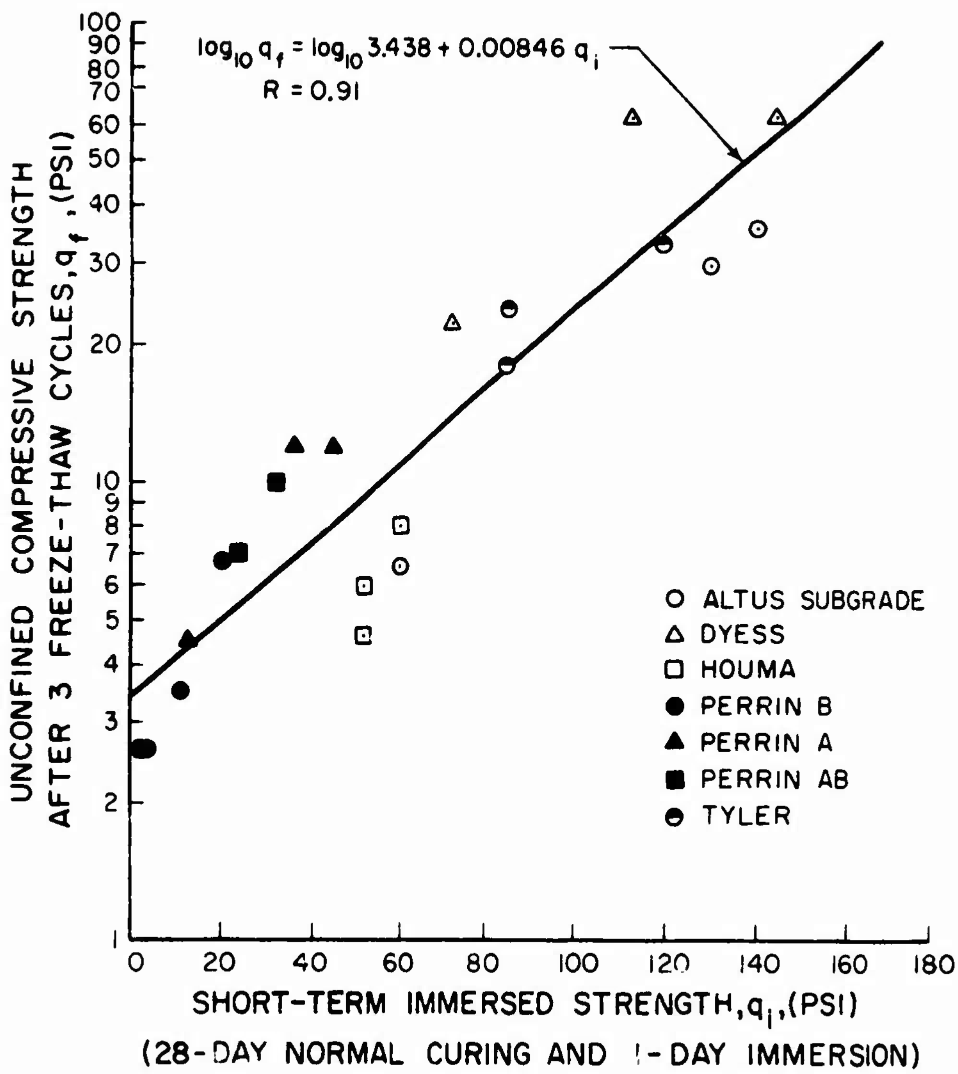


Figure 49 - Correlation Between Residual Strength After 3 Freeze-Thaw Cycles and Short-Term Immersed Strength for Soil-Lime Mixtures

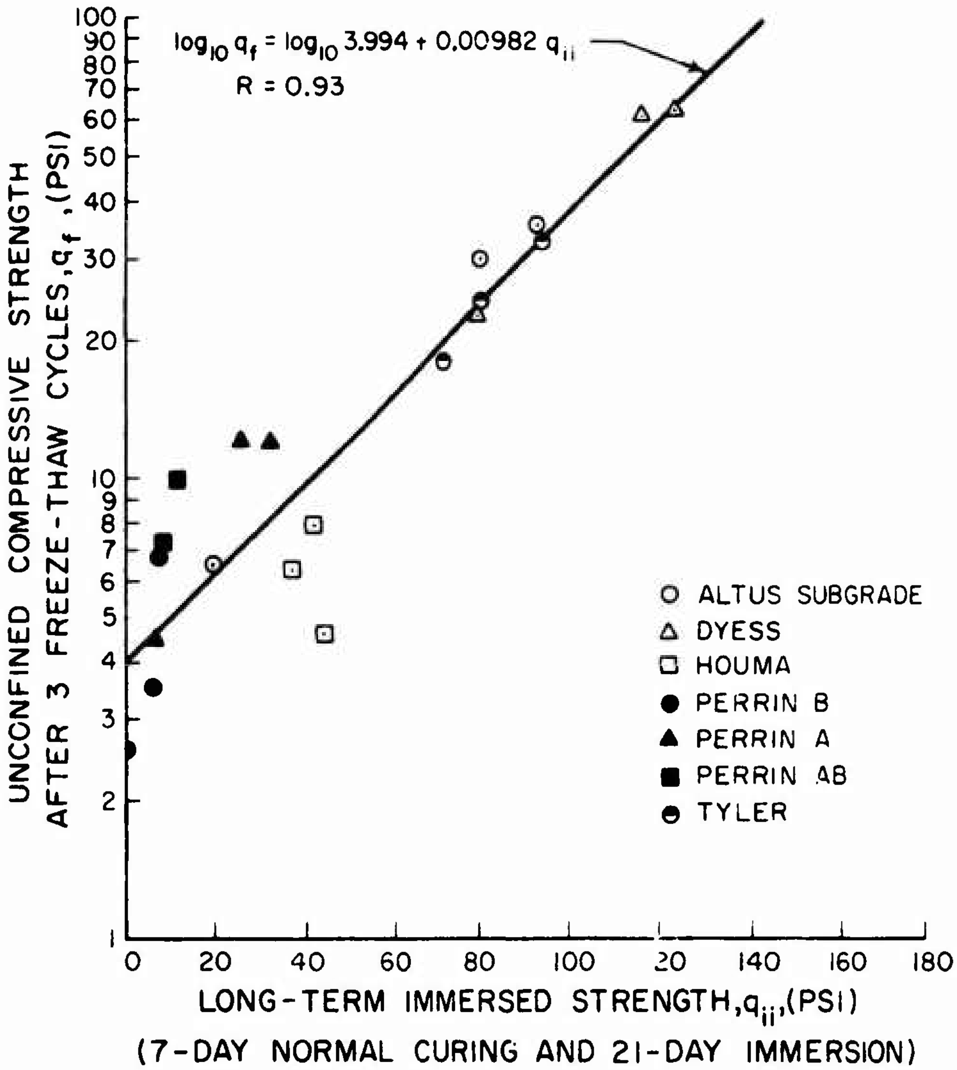


Figure 50 - Correlation Between Residual Strength After 3 Freeze-Thaw Cycles and Long-Term Immersed Strength for Soil-Lime Mixtures

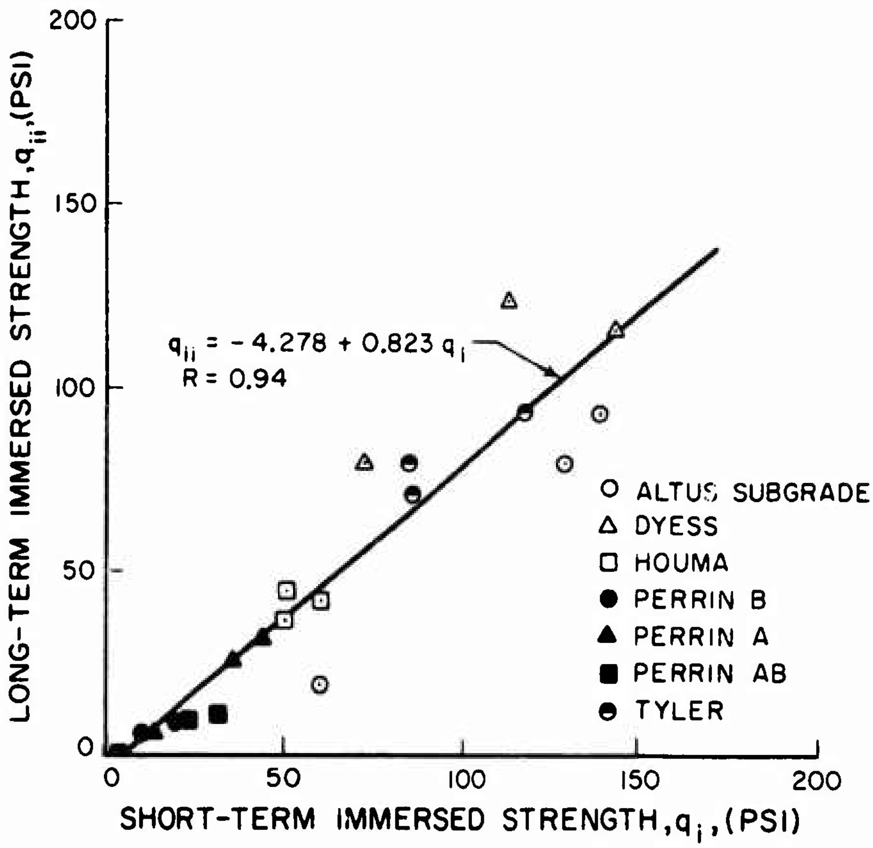


Figure 51 - Correlation Between Long-Term Immersed Strength and Short-Term Immersed Strength for Soil-Lime Mixtures

$$q_{ii} = -4.278 + 0.823\ q_i$$

Correlation coefficient, $R = 0.94$

where: q_i, q_{ii} are as defined above.

Thus, it appears that the results of one type of immersion test can be predicted by knowing the other. Again, it should be realized that this correlation was based on a limited number of soils.

c. Comparison between initial unconfined compressive strengths and durability test results.

It would be convenient if the initial unconfined compressive strengths (28-day strengths) could be correlated with durability of the soil-lime mixtures. However, various comparisons of initial unconfined compressive strength with results of immersion and freeze-thaw tests failed to show a definite relationship. These data are plotted in Figures 52 and 53. The scatter of data is significant; however, a general trend is apparent which may be better defined as more data become available.

A similar comparison was made of the indices of resistance to immersion and freeze-thaw, R_i, R_{ii} and R_f to the initial unconfined compressive strength (Table 5). Plots of these data are not shown, but the results did not show a definite relationship.

11. Short-Cut Test for Evaluation of Durability of Soil-Lime Mixtures

The previous comparison of durability tests indicated that any one durability test could be substituted for the other two. Thus, it appears that the short-term immersion test could be used for rapid evaluation of the durability of soil-lime mixtures.

Short-term immersed strength requirements were developed based on Thompson's (reference 10) data for extended (8-day) soaking. If it is assumed the

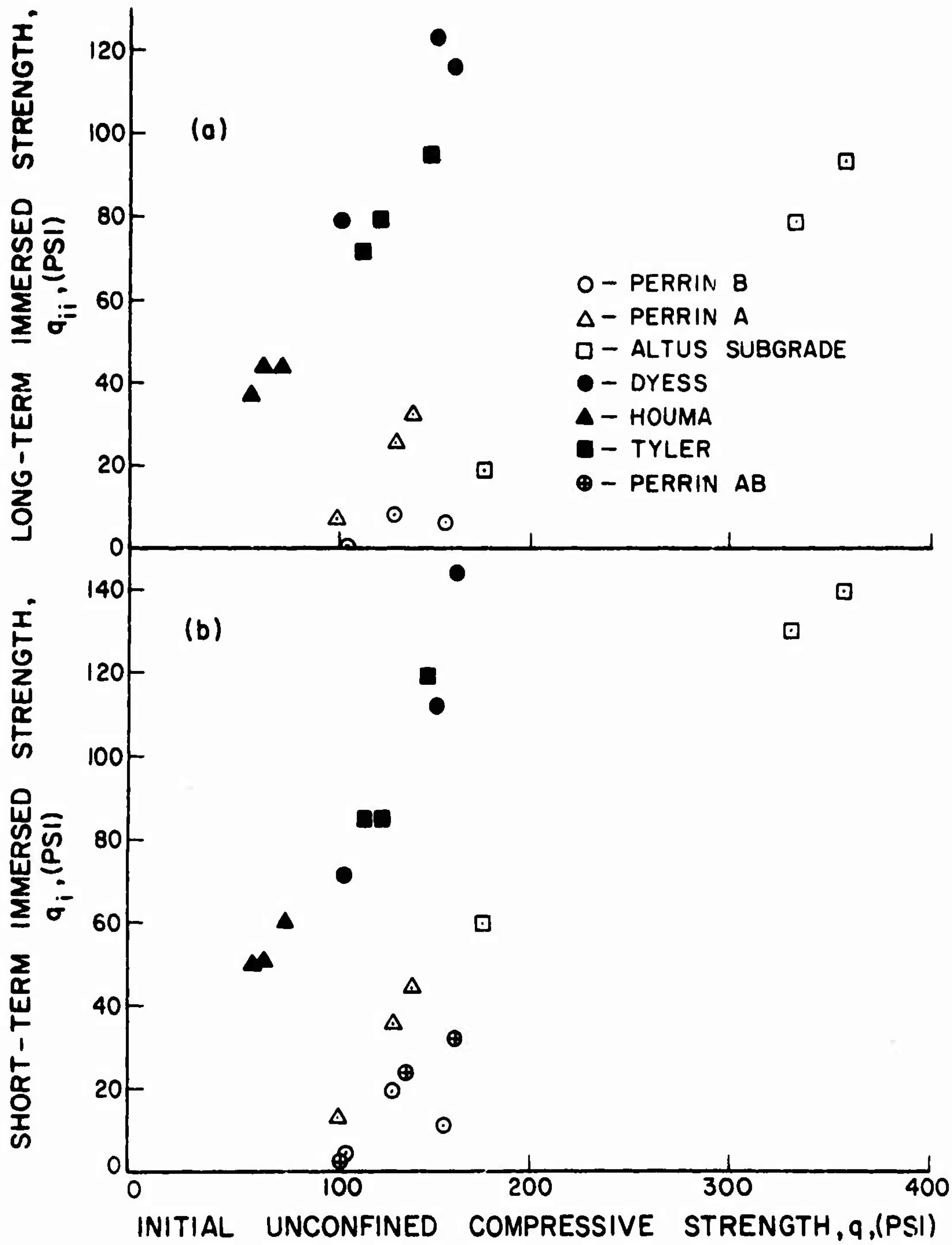


Figure 52 - Relationship Between Immersed Strengths and Initial Unconfined Compressive Strength

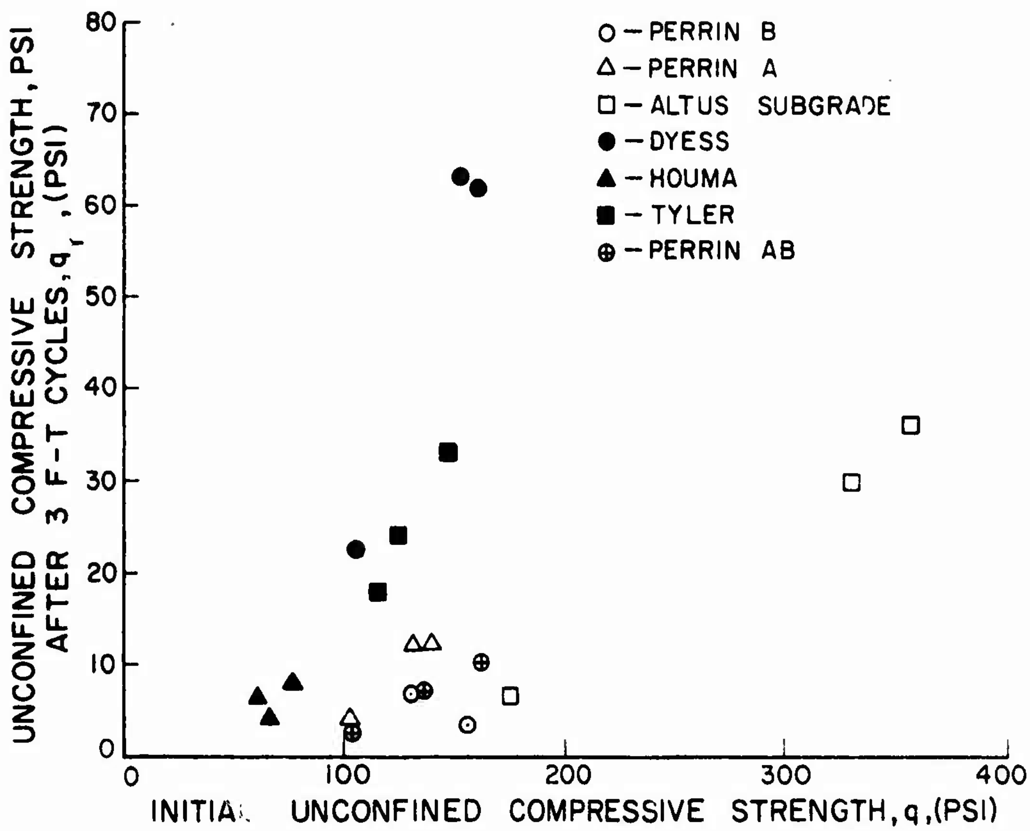


Figure 53 - Relationship Between Residual Strength After 3 Freeze-Thaw Cycles and Initial Unconfined Compressive Strength

residual strengths after soaking 8 days approximate strengths from the long-term immersions used in this research, then Figure 51 can be used as an indirect correlation between Thompson's 8-day soaking and short-term immersion. Table 6 is based on this assumption.

The short-term immersed strengths required to provide adequate residual strength (Table 6) must be considered tentative until the relationship shown in Figure 51 has been verified with many more soils. It is also emphasized that these are minimum quality requirements and are not to be used as design strengths. The design strengths should be evaluated by testing the specific soil-lime mixture being considered.

12. Effects of Sulfates and Organics on Durability

The resistance of lime-stabilized sulfate and/or organic soils against immersion and freeze-thaw cycles was evaluated in terms of the various indices of resistance. These indices were described earlier and are contained in Table 5.

As a first attempt to determine the separate effects of sulfates and organics, the various indices were plotted versus the respective sulfate and organic contents of the soil. Figures 54 and 55 show the effect of sulfate on the indices of resistance to short-term immersion, long-term immersion and freeze-thaw action. The Dyess mixture, which contained no sulfate, exhibited the highest resistance to immersion and freezing and thawing. The Houma mixture had reasonable resistance to long-term and short-term immersion, but the Houma mixture lost a large fraction of its initial strength after being subjected to freeze-thaw cycles. The lower indices of resistance observed for high sulfate bearing soils, Altus subgrade, Perrin B, Perrin A, and Perrin AB, indicated that these mixtures suffered considerable loss from their initial

TABLE 6

TENTATIVE SHORT-TERM IMMERSED STRENGTH

REQUIREMENTS FOR SOIL-LIME MIXTURES

Anticipated Use	Residual Strength Requirement, psi[a]	Short-Term Immersed Strength[b] Requirements
Modified Subgrade	20	30
Subbase		
a. Rigid Pavement	20	30
b. Flexible Pavement		
Thickness of Cover		
10 inches	30	45
8 inches	40	55
5 inches	60	80
Base	100	130

[a]As recommended by reference 10.

[b]Unconfined compressive strength of 2- by 4-inch specimen after 1-day immersion.

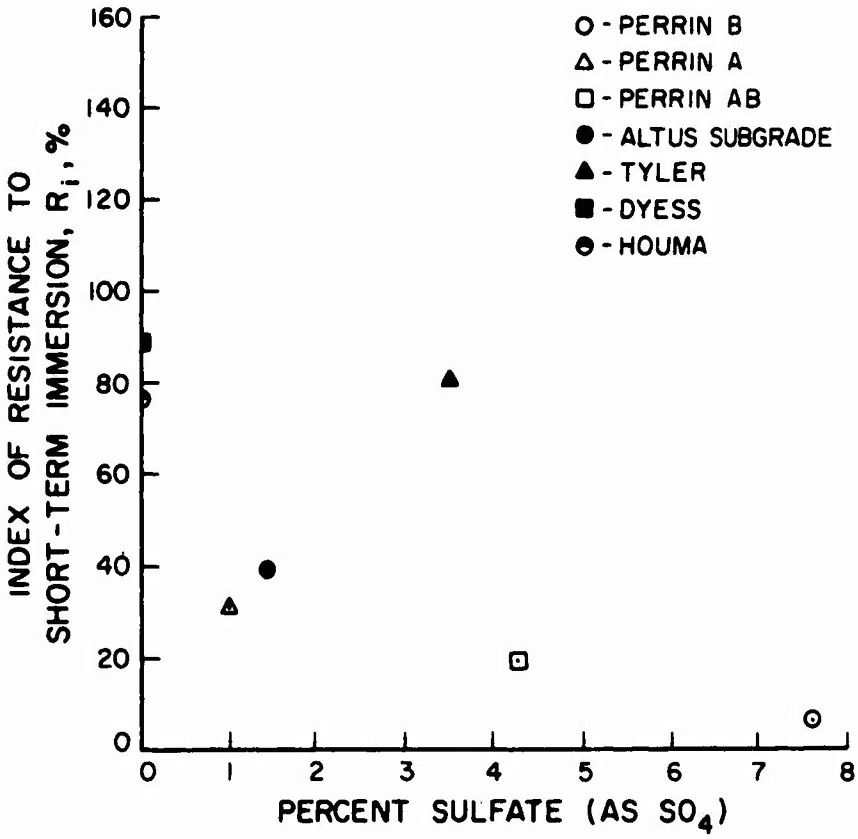


Figure 54 - Effect of Sulfate on Indices of Resistance to Short-Term Immersion for the Soils Treated with Their Respective Strength-Lime Percentages

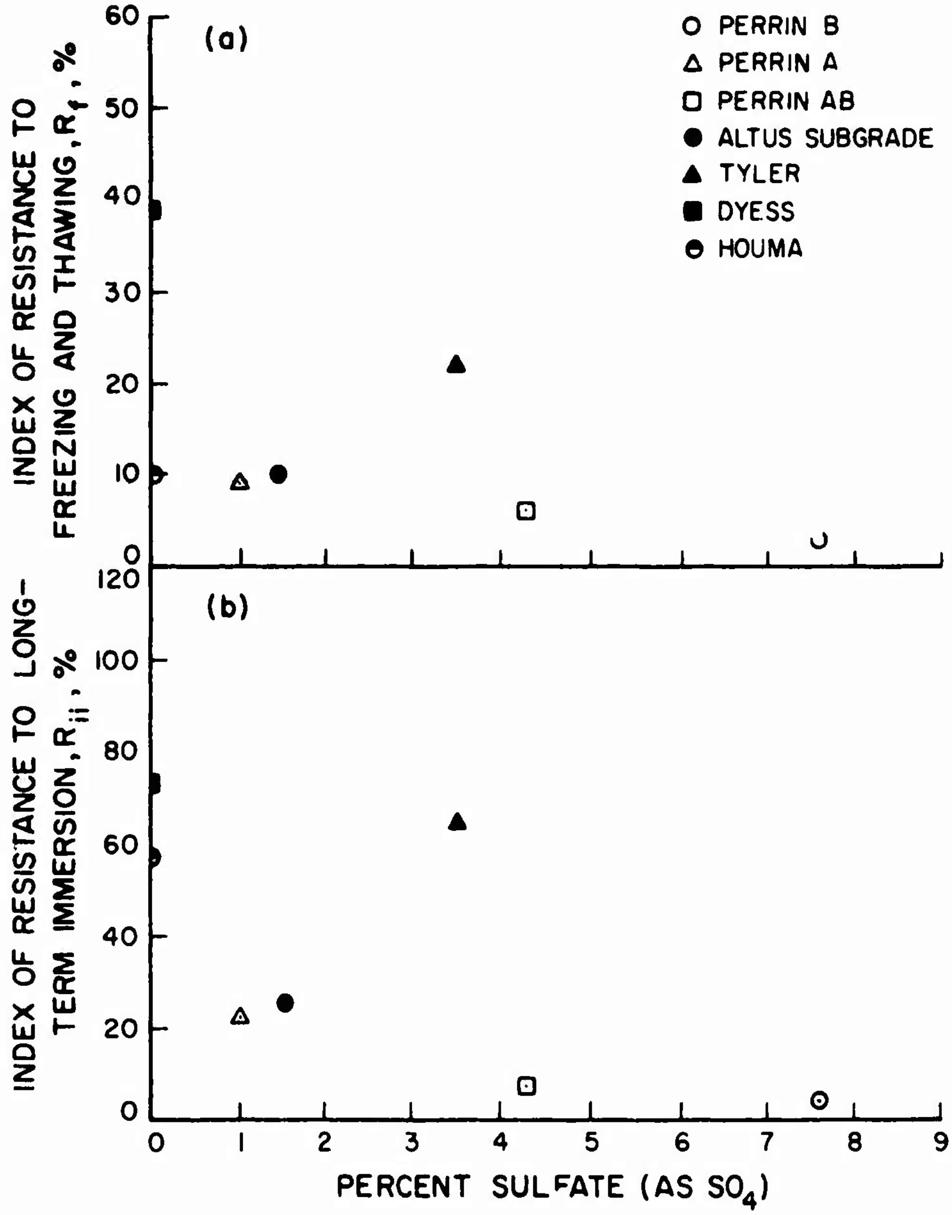


Figure 55 - Effect of Sulfate on Indices of Resistance to Long-Term Immersion and Freezing and Thawing for the Soils Treated with Their Respective Strength-Lime Percentages

strength. The high resistance to immersion and freezing and thawing offered by the Tyler mixture was not fully understood, but it should be recalled that this soil also had a high organic content.

Since both sulfate and organic matter are thought to influence the durability of soil-lime mixtures, it is necessary to eliminate the highly organic soils from consideration before the effect of sulfates can be properly evaluated. If the Tyler and Houma soils were eliminated from Figures 54 and 55, it would appear that a maximum sulfate content of about 0.75 percent could be allowed in a soil before its resistance to immersion or freezing and thawing decreased below an acceptable level.

The relationship between organic content and durability does not appear to be as clear as it was for sulfates. Figures 56 and 57 show that the Dyess mixture, with 1.17 percent organic content, had substantial resistance to immersion and freeze-thaw cycles. The Houma soil (1.43 percent organic matter) displayed considerable resistance to immersion, but it did not have much resistance to freezing and thawing. Perrin B, Perrin A, Perrin AB, and Altus subgrade soils exhibited low resistance to immersion and freeze-thaw cycles, but these soils also contained unacceptable sulfate. The Tyler mixture again behaved differently.

To properly evaluate the effect of organic matter on durability, using Figures 56 and 57, it would be necessary to discard the high sulfate bearing soils, and this would leave insufficient data to analyze.

An alternative method is to use data for all soils and to conduct a statistical analysis which will determine the relative effect of both organics and sulfates on durability. Since there is no physical relationship to rely on, a polynomial of the following form was used:

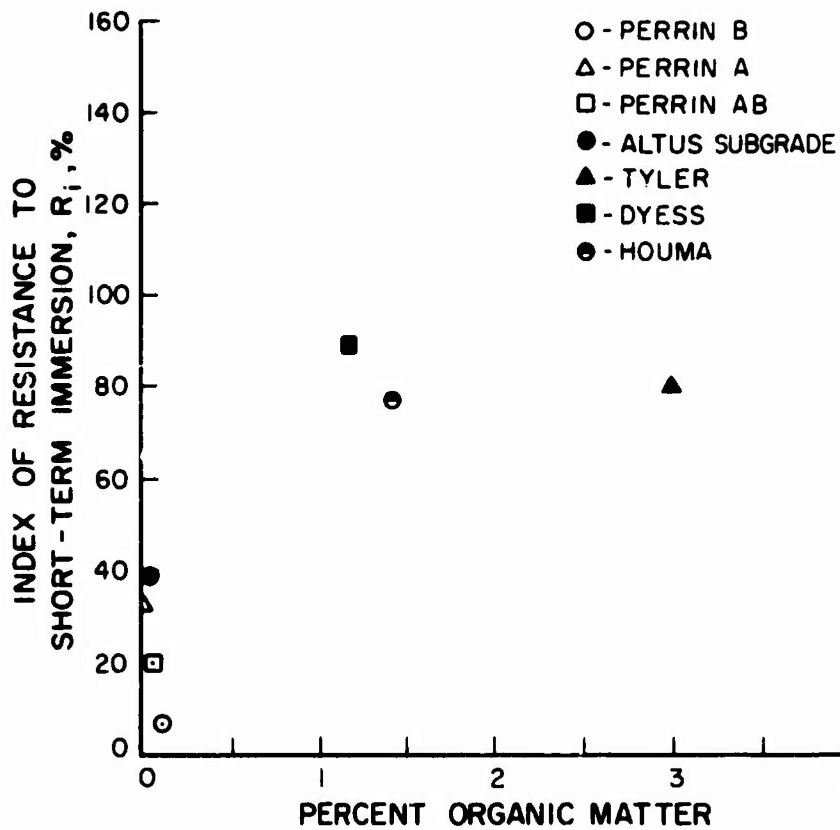


Figure 56 - Effect of Organic Matter on Indices of Resistance to Short-Term Immersion for the Soils Treated with Their Respective Strength-Lime Percentages

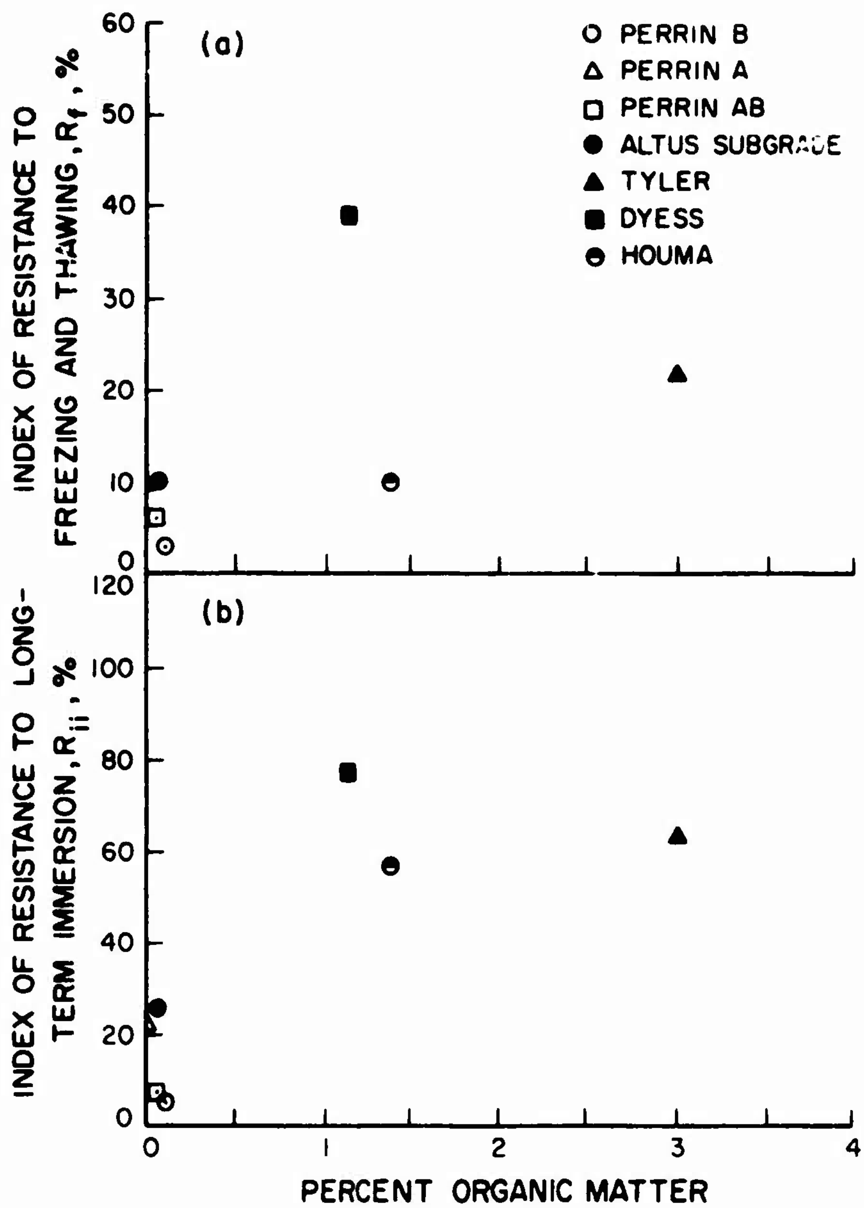


Figure 57 - Effect of Organic Matter on Indices of Resistance to Long-Term Immersion and Freezing and Thawing for the Soils Treated with Their Respective Strength-Lime Percentages

$$Y_i = A_0 + A_1(X1) + A_2(X2) + A_3(X1)^2 + A_4(X2)^2 + A_5(X1)(X2)$$

where Y_i = appropriate index of resistance, R_i, R_{ii} or R_f

X1 = sulfate content in soil

X2 = organic content in soil

A multiple least squares regression analysis was performed on a digital computer using all soils and each index of resistance. The program used employs the Hocking-LaMott-Leslie step-down regression technique. In this method the dependent variable is regressed on appropriate combinations (or subsets) of independent variables by a step-down procedure, and, thus, the optimum combination of independent variables is obtained. Based on the correlation coefficients and the probability levels, the user can select the most appropriate analysis. The most suitable analysis for each index of resistance is:

Dependent Variable	A_0	A_1	A_2	A_3	A_4	A_5	R^2	Standard Error
R_i	43.4	- 9.1	100.5	--	-53.1	23.4	.996	3.7
R_{ii}	31.1	-10.0	134.1	--	-79.6	36.3	.997	2.9
R_f	5.2	--	134.3	-1.0	-90.4	41.9	.924	6.1

Both the graphical and statistical analyses described above are somewhat complicated by the Tyler soil, which performed better than its high organic and sulfate contents indicated. There is no reason to doubt the response (indices of resistance) of the Tyler soil to the various durability tests, but it is possible that the determination of the sulfate content was influenced by the presence of the high quantity of organics, or vice versa. With this consideration in mind, the statistical analyses were performed again without the Tyler data. The results are:

Dependent Variable	A_0	A_1	A_2	A_3	A_4	A_5	R^2	Standard Error
R_1	39.0	- 6.1	119.4	--	-64.9	--	.999	1.4
R_{i1}	32.3	-10.9	140.6	0.7	-85.1	--	.999	1.2
R_f	8.9	- 2.6	140.0	--	-96.8	--	.970	3.8

Neither of the above analyses is considered suitable for general use, or for extrapolation to other soils, owing to the limited amount of data available for the analyses. Rather, the primary use is to indicate trends and to observe the general effects of sulfates and organics on the durability of soil-lime mixtures. For example, the statistical analyses with or without the Tyler soil indicate coefficients (A_1) for the sulfate content. This means that the indices of resistance will decrease as sulfate content increases. Conversely, the organic content always had positive coefficients, but in every case a negative coefficient was associated with the square of the organic content. This indicates that there may be an optimum organic content for durability, and that the optimum amount of organics is influenced somewhat by the sulfate content. Additional data are needed to test these concepts.

13. Relationship Between Lime Reactivity and Initial Unconfined Compressive Strength

An analysis was made to determine whether a relationship existed between lime reactivity (Δq) and the initial unconfined compressive strength (28-day strength). A plot of the experimental data is shown in Figure 58. As additional verification of this relationship, data previously reported (reference 17) by Thompson were also plotted (Figure 59) and analyzed. These relationships were developed for lime-treated soils compacted at optimum moisture content and AASHO T-99 density, with a subsequent 28-day normal curing period with no change in moisture allowed.

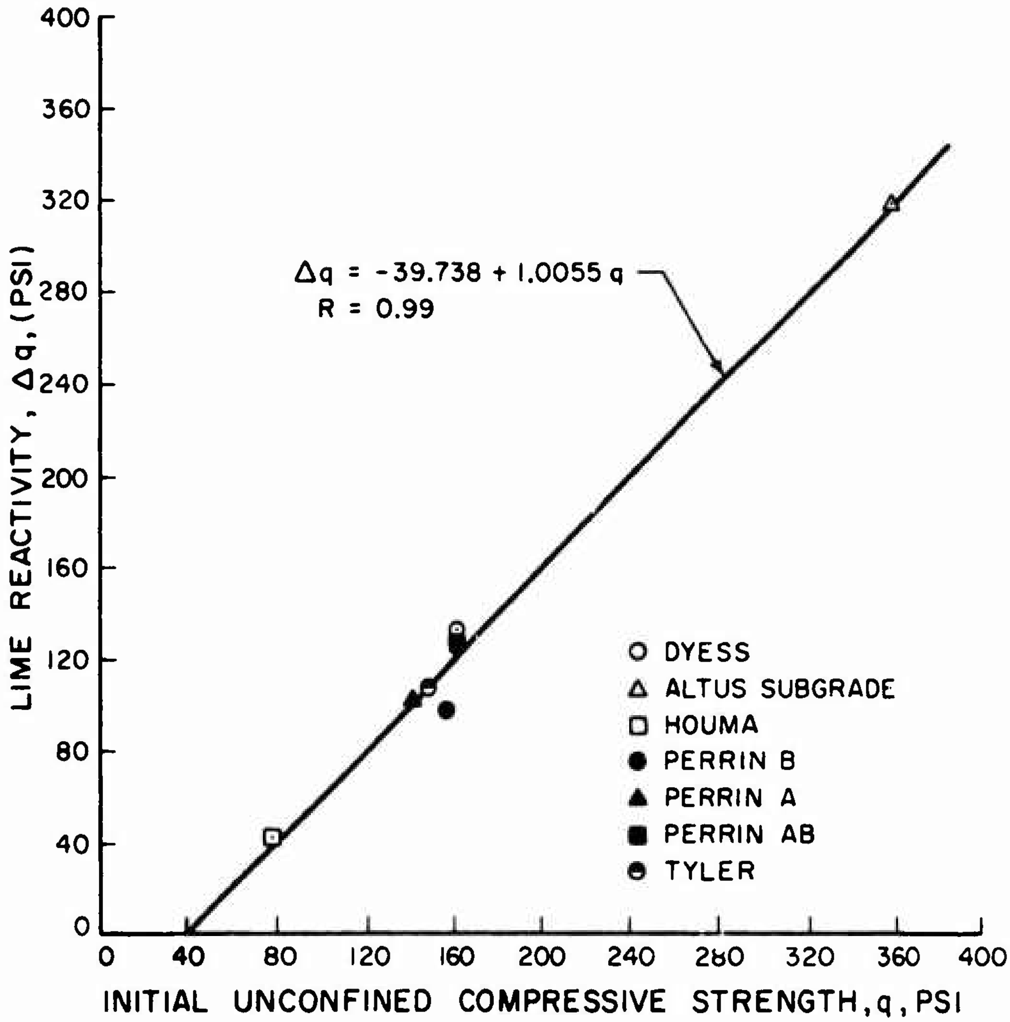


Figure 58 - Relationship Between Lime Reactivity and Initial Unconfined Compressi'e Strength

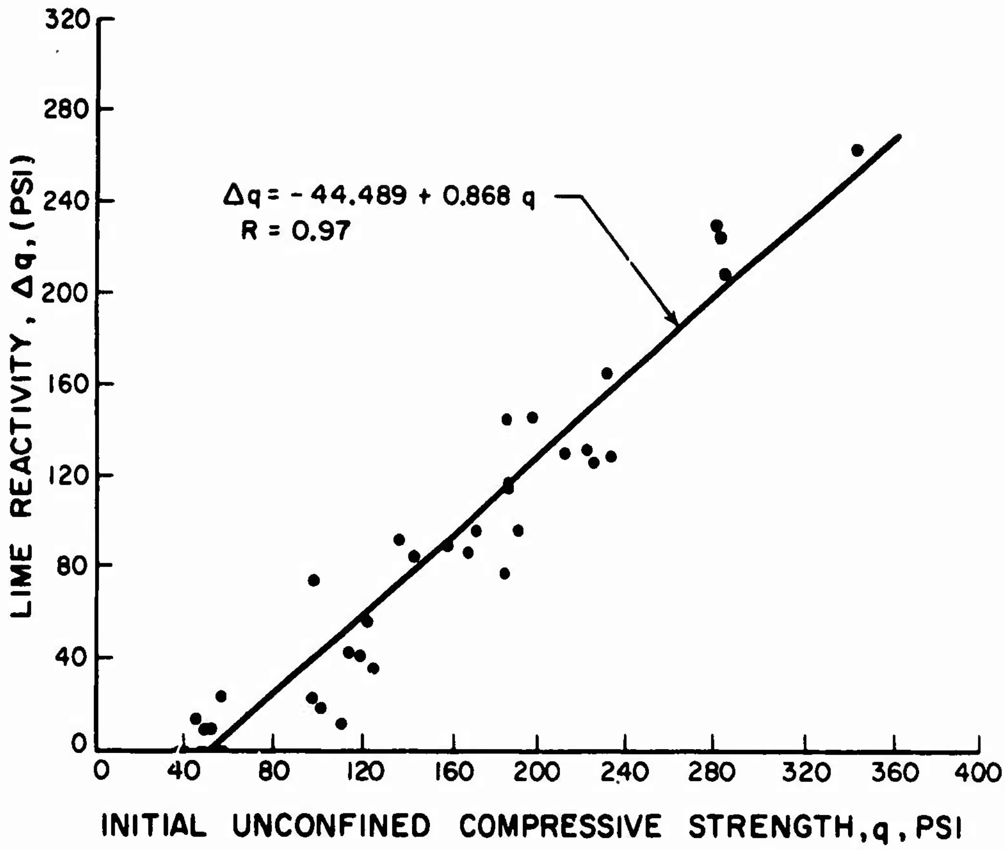


Figure 59 - Relationship Between Lime Reactivity and Initial Unconfined Compressive Strength

The relationship from the experimental data compared favorably with that obtained using Thompson's test data. Table 7 shows the intercepts and slopes of the linear regression equations obtained from each set of data.

Figures 58 and 59 show that a soil-lime specimen whose initial unconfined compressive strength is less than 90-100 psi (depending on which set of data is considered) will not be lime reactive according to the definition of lime reactivity (strength increase of 50 psi). For safety, a minimum strength of 110 psi should be used.

This conclusion should be used with caution since the supporting data are rather limited.

TABLE 7

SLOPES AND INTERCEPTS OF LINEAR REGRESSION EQUATIONS USING THE EXPERIMENTAL DATA AND THOMPSON'S TEST DATA

Data Used	Slope	Intercept
Experimental	1.006	-39.74
Thompson	0.870	-44.49

14. Relationship Between pH-Lime Percentage, Strength-Lime Percentage and Fixation-Lime Percentage

This study was aimed at determining whether a relationship existed between the optimum lime contents determined by the three types of tests. Although the strength-lime percentage is usually the one desired in soil stabilization work, the equipment and time necessary to determine the strength-lime percentages makes it desirable to have a more expedient method of obtaining this lime content. The optimum lime percentages determined by the three different test procedures are shown in Table 8 for the various soils tested. It should be noted that a strength-lime percentage was not established for Tyler

TABLE 8

CORRELATION BETWEEN FIXATION-LIME, pH-LIME AND STRENGTH-LIME PERCENTAGES

Soil	Lime Fixation Point, %	Fixation-Lime %[1] (Lime Fixation 2%)	pH-Lime %2	Strength-Lime %3
Perrin B	4	6	6	5.5
Perrin A	6	8	8	8.5
Perrin AB	6	8	8	>8.5
Altus Subgrade	3.5	5.5	5	6
Dyess	2	4	4	4
Houma	4	6	5	5
Tyler	>12	>14	12	>14

[1,2,3]As defined in Appendix L.

or Perrin AB soils, nor was a fixation-lime percentage developed for the Tyler soil.

Figure 60 compares the strength-lime percentage with the pH-lime percentage. With the exception of the Tyler and Perrin AB soils the strength-lime and pH-lime percentages compared to within ± 1 percent lime content. The strength-lime and fixation-lime percentage also compared to within ± 1 percent lime content (Figure 61), except for the Tyler and Perrin AB soils. The common factor in these two soils is a high sulfate content, and a preliminary conclusion, based on the limited soils encountered herein, is that there is no short-cut procedure for determining strength-lime percentage in soils containing high sulfate contents.

Thompson and Eades (reference 21) demonstrated that the pH test procedure gave somewhat conservative results, i.e., it indicated lime contents somewhat higher than those determined by strength tests. There is no indication that this is true based on the limited number of soils investigated in this research. For the Perrin AB and Tyler soils the pH test gave results which were unconservative by at least 0.5 percent and 2 percent respectively; however, these are admittedly rare soils. The largest deviation among the remaining soils occurred with the Altus subgrade, another soil with relatively high sulfate content, where the pH test gave results which were 1 percent too low. However, for Perrin B, the highest sulfate-bearing soil, the pH test was conservative by 0.5 percent lime. Thus for the soils investigated in this research, there is no clear-cut indication that the pH test is conservative or unconservative, although it gave unconservative results on three out of the four high sulfate-bearing soils.

The pH test does not necessarily indicate that a soil will be stabilized if the pH-lime percentage is added to it. For example, the Houma soil did not

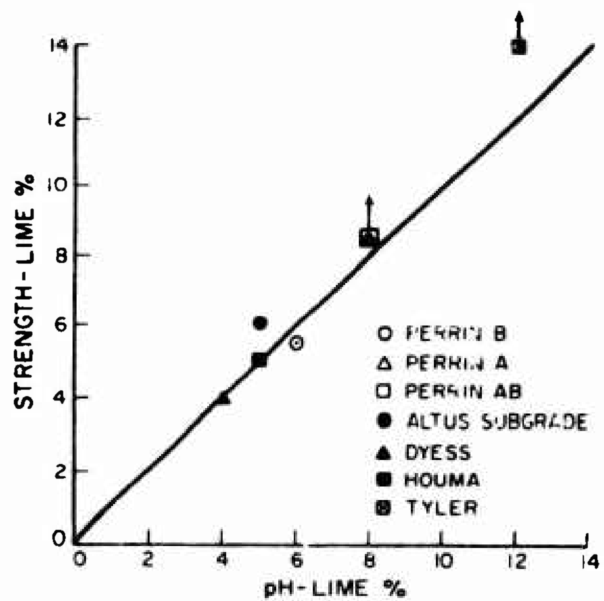


Figure 60 - Relation Between Strength-Lime Percentages and pH-Lime Percentage

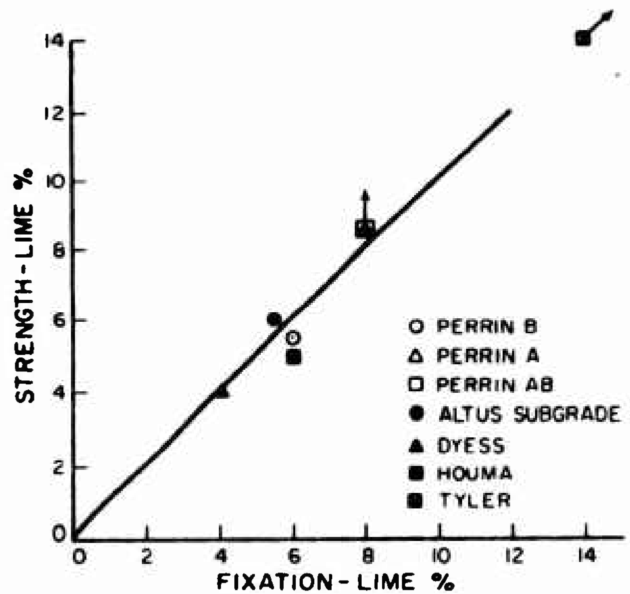


Figure 61 - Relation Between Strength-Lime Percentages and Fixation-Lime Percentage

react with lime to achieve a substantial strength increase although the soil was modified in many other ways such as reduced plasticity, reduced swell, improved workability, etc. Since strength improvement is the primary objective of stabilization, strength tests should be performed in conjunction with the pH test.

For expedient construction purposes, the pH test could be used to estimate the lime content as accurately as any other hasty method, but the strength of the soil would need to be determined to see if the soil was stabilized or merely modified. This could be done hastily be determining unconfined compressive strengths of accelerated cured specimens (reference 15). For non-expedient construction, the pH test could be used to estimate the lime requirement, but compressive strength and durability tests should follow.

Figure 14 showed that a good correlation existed between the pH-lime percentage and the fixation-lime percentage. On the basis of the limited soils investigated herein, either test could be used for determining the lime content to be used, and both tests have about the same limitations. However, the pH test is recommended because it is quicker and requires less skill to perform.

15. Modification to Lime Stabilization Subsystems

On the basis of the preceding information, it appeared that changes should be made in the original lime stabilization subsystems to utilize the advantages of accelerated curing and accelerated durability testing.

It should be realized that the tests for determining sulfates and organics are not within the realm of expedient testing. Furthermore, they are not standard soil engineering tests that most laboratories are prepared to perform. In addition, the application of the test results to lime stabilized soils requires additional verification and setting of limits before confidence can be

reached. For this reason, it is suggested that durability tests be utilized for all except the most expedient situation. Using the correlations developed between various durability tests, estimates of durability can be made even under rather severe time constraints.

Once the soil has been considered for lime stabilization, there are three basic steps that must be performed.

a) Estimate the lime content to be used

b) Determine the strength (or lime reactivity) of the soil-lime mixture

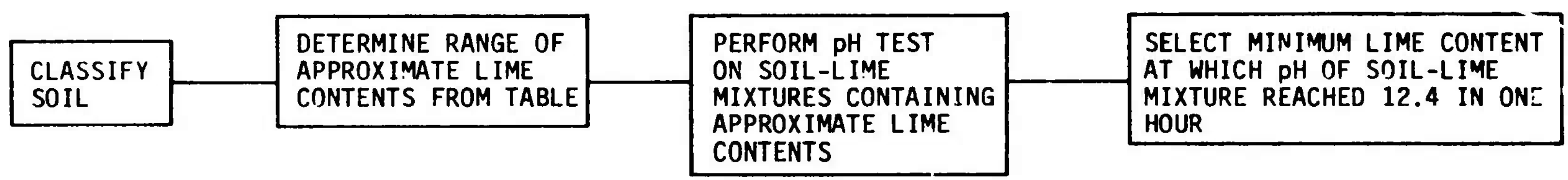
c) Determine the durability of the soil-lime mixture.

Steps b) and c) should be performed at several lime contents centered around the estimated lime content. Thus, the logical fourth step would be:

d) Select the optimum lime content.

The hastiest decisions to be made will probably be in the expedient subgrade situation where a decision must be made in a day or so. Under this situation, little more can be hoped for than to estimate the optimum lime content (pH test) without regard to whether this soil will react with lime. Somewhat more time should be available for the expedient base course situation, thereby allowing the use of accelerated curing procedures to prepare samples for determining lime reactivity. In situations where the time factor is not so stringent, expedient subgrades can be examined according to the expedient base course procedures. Figures 62 and 63 show the revised procedures.

For the nonexpedient situations, it is assumed that adequate time will be available for all phases of testing. Both lime reactivity and durability of the treated mixtures can be examined. This is shown by the revised procedures for nonexpedient situations presented in Figures 64 and 65.



NOTE: If adequate time is available, the procedure for expedient base course should be used.

Figure 62 - Subsystem for Expedient Subgrade Stabilization with Lime

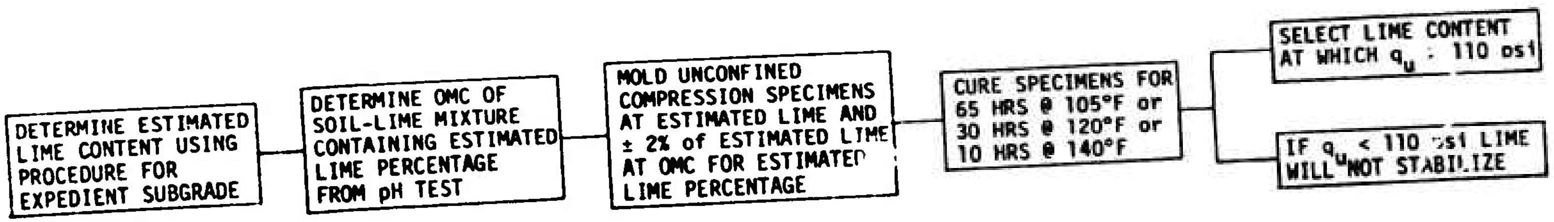


Figure 63 - Subsystem for Expedient Base Course Stabilization with Lime

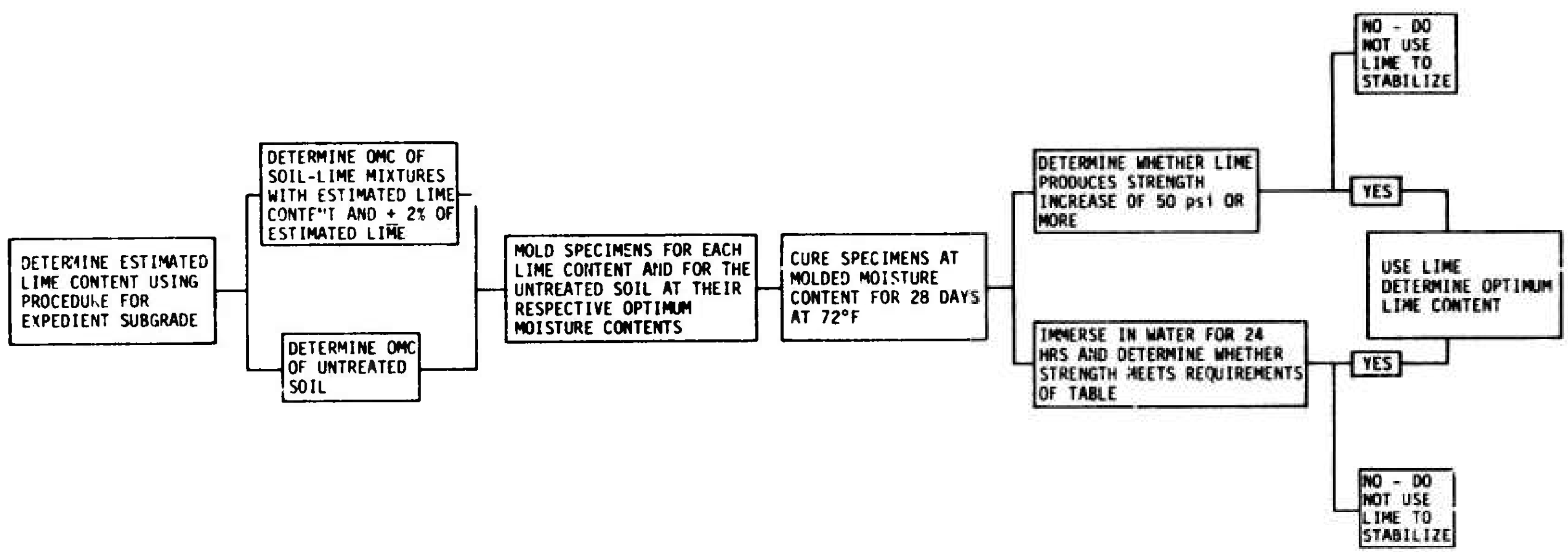


Figure 64 - Subsystem for Nonexpedient Subgrade Stabilization with Lime

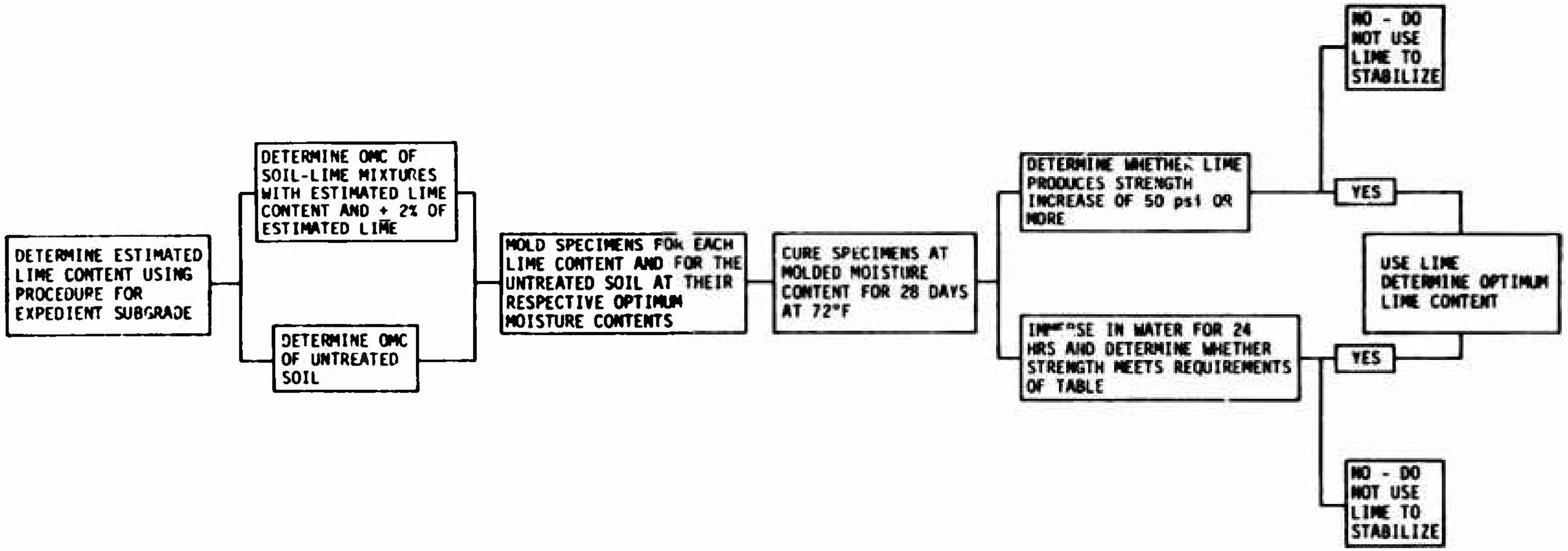


Figure 65 - Subsystem for Nonexpedient Base Course Stabilization with Lime

SECTION IV

CEMENT STABILIZATION

1. Present System and Research Needs

As with lime, plasticity index is also used in the process of determining whether cement should be used for stabilizing a soil (Figures 2, 3, 4 and 5). Cement may be an effective stabilizer for low plasticity soils where it acts primarily as an agent to cement the soil grains together without involving significant chemical reaction with the grains. With high plasticity fine grained soils there is a reaction similar to that between soil and lime. Soil-cement can not, therefore, be regarded as a simple mixture of hydrated cement particles bonding together unaltered clay particles, but should be considered as a system in which both clay and hydrating cement combine through secondary reactions producing additional cementitious materials. However, if the plasticity index exceeds about 30, the cement becomes difficult to mix with the soil. Lime should be added to reduce the plasticity index and condition the soil prior to receiving the cement.

The subsystems given in Figures 66 through 69 are utilized in SSIS to determine whether cement will stabilize the soil and how much cement is required. These subsystems are based, to a large degree, on procedures developed by the Portland Cement Association. There are, however, two tests -- the test for sulfate content and the pH test for organics -- that are nonstandard.

The effect of organic matter on cement stabilized soils is closely associated with its ability to combine with calcium ions liberated by hydrating cement. The adsorption of calcium ions from the hydrating cement by the active organic matter results in reduction of pH of the soil-cement-water system.

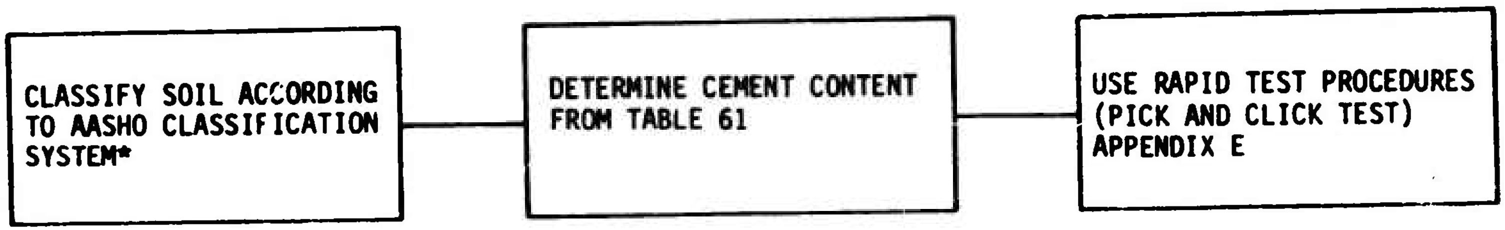


Figure 66 - Subsystem for Expedient Subgrade Stabilization with Portland Cement (from reference 4)

(NOTE: Table and Appendix Nos. refer to those in original reference)

*Although the unified classification system can be used, the AASHO is preferred.

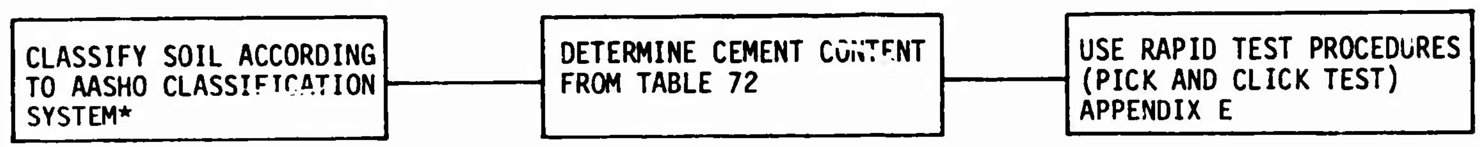


Figure 67 - Subsystem for Expedient Base Course Stabilization with Portland Cement (from reference 4)

(NOTE: Table and Appendix Nos. refer to those in original reference)

*Although the unified classification system can be used, the AASHO is preferred.

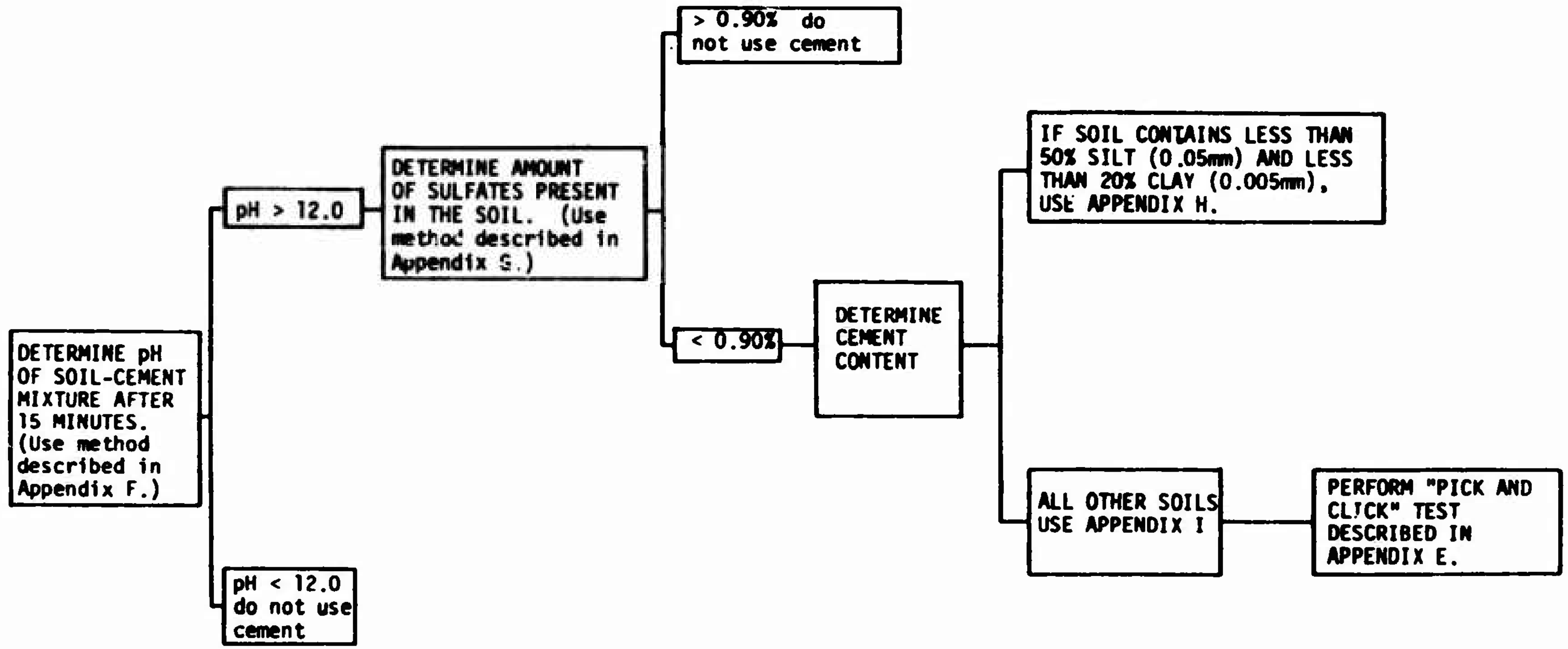


Figure 68 - Subsystem for Nonexpedient Subgrade Stabilization with Cement (from reference 4)

(NOTE: Table and Appendix Nos. refer to those in original reference)

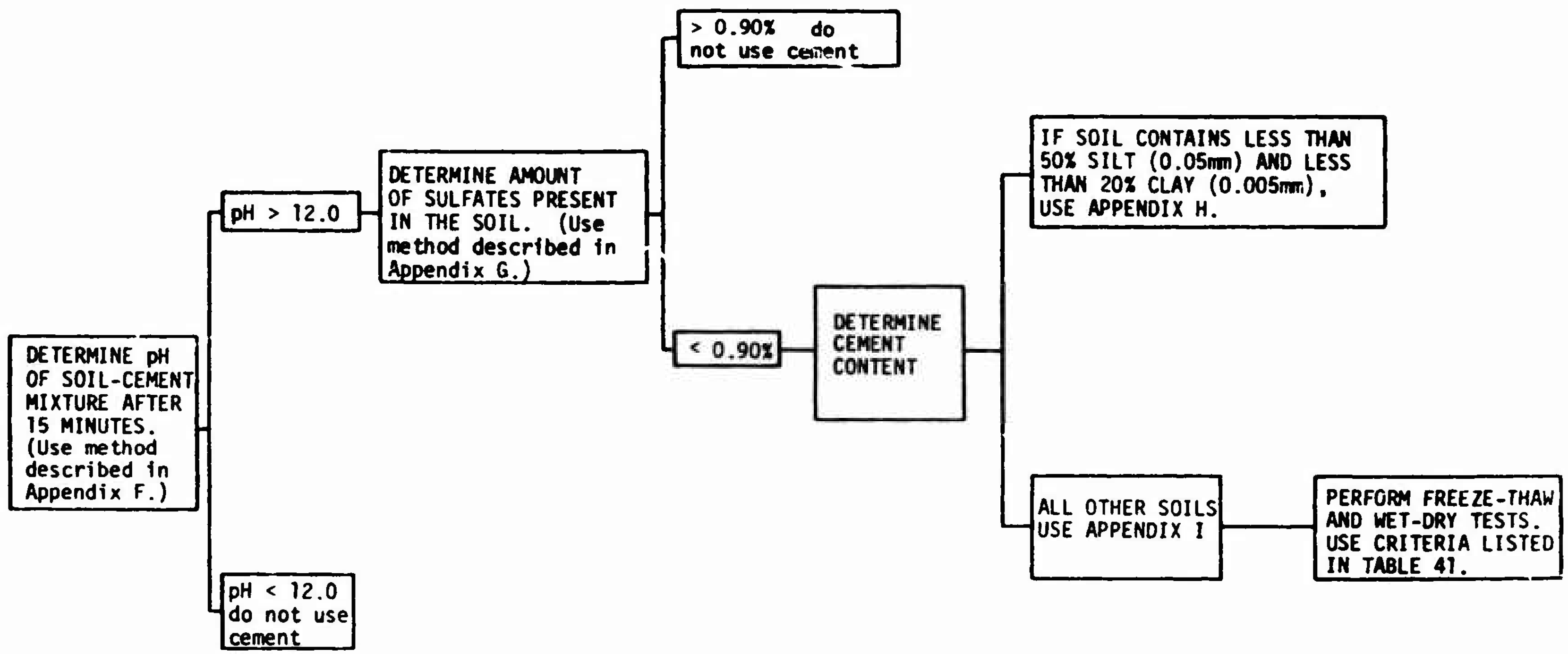


Figure 69 - Subsystem for Nonexpedient Base Course Stabilization with Cement (from reference 4)

(NOTE: Table and Appendix Nos. refer to those in original reference)

The effect of the reduced pH is to cause the precipitation of an alumina-silica gel over the cement particles, thus inhibiting the normal hardening process (reference 22). The pH test utilized in SSIS involves the determination of the pH of a 10:1 by weight mixture of soil and cement 15 minutes after mixing. If the pH is at least 12.0, it is assumed that organics, if present, will not interfere with normal hardening of the soil-cement.

Sulfate in soil-cement can result in disintegration of the stabilized soil when immersed in water, but it apparently has little effect on strength of mixtures cured at constant moisture content and not wetted above this moisture content (reference 23). In SSIS, the maximum allowable value of sulfate was set at 0.9 percent as determined by either of two methods presented.

The pH test for organics is relatively simple and involves no major outlay of time. It was only the numerical limit of pH that required verification. On the other hand, the sulfate test is complicated, and it would be worthwhile to delete this test if it were not necessary. If found to be necessary, then the allowable limit of sulfate should be verified.

In addition the required criteria as set forth by PCA needed to be verified or adjusted, if necessary.

Testing involved determination of 7- and 28-day compressive strengths, wet-dry and freeze-thaw durability tests, as well as the pH and sulfate determinations.

2. pH Test and Sulfate Determination

The diagnostic pH test proposed by Maclean and Sherwood (reference 22) for detecting the presence in soil of organic matter interfering with the hardening of soil-cement was performed on soil-cement pastes of all soils. The test procedure is detailed in Appendix F. Test results are shown in Table 9. There is no difficulty in interpreting the results of this pH test.

TABLE 9

RESULTS OF pH AND SULFATE TESTS

Soil	pH (10:1)	Sulfate Percent
Perrin B	12.10	7.64
Perrin A	12.16	1.00
Perrin AB	12.10	4.30
Altus Subgrade	12.05	1.52
Dyess	12.25	0.00
Houma	11.90	0.00
Tyler	9.25	3.50
Tuy Hoa	10.85	0.00
Altus Subbase	12.07	0.09

Sulfate determinations were made using the test procedure given in Appendix G - Turbidimetric Method. As mentioned earlier, this test is difficult to perform, and it is not a standard test used in the soil engineering field. Table 9 contains the test results.

3. Preparation of Specimens for Strength and Durability Tests

Following the procedure originally outlined in SSIS, the estimated cement content for each soil was determined using the Portland Cement Association recommendations (reference 24). The results are given below:

Soil	Estimated Cement Content
Tuy Hoa	10 percent
Altus subbase	7 percent
Dyess	12 percent
Altus subgrade	13 percent
Tyler	13 percent
Houma	13 percent
Perrin A	13 percent
Perrin B	13 percent

Moisture-density tests were performed on soil-cement mixtures containing the estimated cement content in accordance with ASTM Designation D 558-57. Three trial cement contents were then selected to encompass the estimated cement content, and Proctor specimens were molded for each cement content at the optimum moisture content obtained for the estimated cement content. These specimens were cured in a moist room at 73°F (±2) and 100 percent relative humidity.

Unconfined compression tests were performed on specimens molded at the estimated cement content after being normally cured for 7 days and 28 days in the moist room. These specimens were soaked in water for 4 hours before they were tested. Unconfined compression tests were also performed on the specimens after being subjected to long-term immersion (7 days normal curing followed by 21 days immersion).

Specimens molded at all three cement contents were subjected to freeze-thaw (F-T) and wet-dry (W-D) durability tests according to ASTM Designations D 560-57 and D 559-47 with the exception that length changes were made rather than volume changes. For these measurements a length comparator equipped with a dial gage graduated to 0.001 inch was used. To attach reference points that would hold throughout the test, an epoxy resin was used to cement a 1/4-inch diameter glass ball to the top of the specimen. Length changes, moisture contents and weight losses after brushing of the specimens were determined at the end of 0, 4, 8 and 12 F-T or W-D cycles. Unconfined compression tests were performed at the end of 12 cycles only on those specimens that were molded at the estimated cement content.

4. Results of Strength Tests

The generalized strength criteria used for determining suitability of soil with respect to cement stabilization are: (1) the soil when treated with cement should have significant unconfined compressive strength after 7-day normal curing, and (2) the strength should further increase after being normally cured for 28 days.

For evaluation purposes, an average 7-day strength of 350 psi may be considered as a minimum requirement for design of soil-cement bases and subbases.

Tuy Hoa, Altus subbase, Dyess and Altus subgrade, when treated with their respective median cement percentages, displayed 7-day unconfined compressive

strengths above 350 psi (Figure 70). The high 7-day strengths of these mixtures and their subsequent strength gains after 28-day normal curing indicated that these soils were suitable for cement stabilization. Tyler, Houma, Perrin B and Perrin A mixtures had low 7-day strengths; they did not exhibit significant strength gains when normally cured from 7 days to 28 days, and could not be considered as stabilized. The low unconfined compressive strength of the Tyler soil was apparently due to the presence of large amounts of sulfate and organic matter. It is believed that Houma, Perrin A and Perrin B soils could not be stabilized with cement partly due to the presence of organic matter (Houma) or sulfate (Perrin A and Perrin B), and partly due to high plasticity. The detrimental effect of sulfate and organic matter on cement stabilization of soils has been previously discussed. High plasticity interferes with cement stabilization because the plastic soils can not be broken down into sufficiently small aggregations to achieve a satisfactory microdistribution of cement.

The fact that those soils having a plasticity index greater than 30 (Perrin A, Perrin B and Houma) were not stabilized with cement was in agreement with the original SSIS criteria shown in Figures 2, 3, 4 and 5.

5. Results of Freeze-Thaw Test

Weight loss, residual strength, unit length change and moisture gain were the methods used for evaluating freeze-thaw (F-T) durability of soil-cement mixtures.

a. Weight loss and residual strength.

Figure 70 shows residual strengths after 12 cycles of freezing and thawing for specimens molded at the estimated cement content. Table 10 shows residual strengths and weight losses for all of the soil-cement mixtures. The PCA

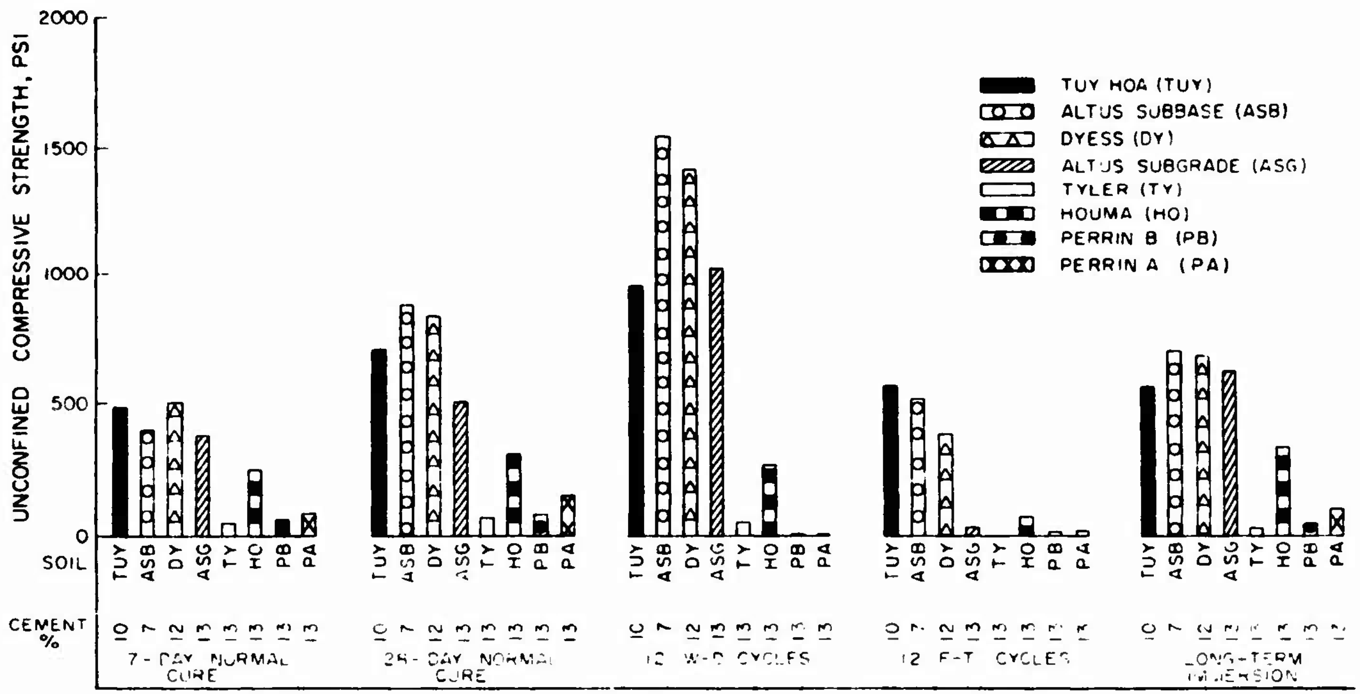


Figure 70 - Comparison of Unconfined Compressive Strengths of Soils Treated with Median Cement Percentages

TABLE 10

SUMMARY OF FREEZE-THAW TEST RESULTS OF SOIL-CEMENT MIXTURES

	Percent Cement	7-Day Strength q_7, psi	28-Day Strength q, psi	q_7 %	Percent After 12 F-T Cycles[a]	Unconfined Compressive Strength After 12 F-T Cycles, q_f, psi — Specimen 1	Specimen 2	Average	R_f %	Unit Length Change After 12 F-T Cycles in/in ($\times 10^{-4}$)	Water Content of Molded Specimens Percent	Water Content of Specimens After 12 F-T Cycles Percent	Recommended Percent Cement
Tuy Hoa	6	166.3	215.5	129	2.94	245.1	-	245.1	113				
A-1-b	10	492.6	693.9	140	0.94	705.1	413.8	559.5	80	48.0	9.3	12.0	10
	13	545.9	961.3	176	0.34	727.3	749.6	738.5	76	-	-	-	
Altus	5	279.5	550.7	197	11.56	254.6	248.3	251.5	45	-	-	-	
Subbase	7	406.4	875.4	215	4.14	417.0	529.2	473.1	54	19.7	10.4	10.4	7
A-2-4	9	493.4	977.2	198	2.12	678.0	732.1	705.1	72	-	-	-	
Dyess	10	-	-	-	4.12	-	-	-	-	-	-	-	
A-7-6(12)	12	506.9	823.5	162	2.70	450.4	324.7	287.5	47	94.0	19.9	23.7	12
	14	-	-	-	0.30	-	-	-	-	-	-	-	
Altus	11	-	-	-	49.23+	-	-	-	-	-	-		Not
Subgrade	13	383.2	507.6	132	45.41+	37.4	17.5	27.5	5	1170	22.2	-	Recommended
A-7-6(12)	15	-	-	-	32.98+	-	-	-	-	-	-		
Tyler	11	-	-	-	50.27+	-	-	-	-	-	-		Not
A-7-5(15)	13	51.7	66.9	129	36.24+	0[b]	0[b]	0	0	ND[c]	30.3	ND	Recommended
	15	-	-	-	28.23+	-	-	-	-	-	-		
Houma	11	-	-	-	38.62	-	-	-	-	-	-		Not
A-7-6(20	13	254.5	313.1	123	35.03	74.8	62.9	68.9	22	573	29.5	39.0	Recommended
	15	-	-	-	19.42	-	-	-	-	-	-		
Perrin B	11	-	-	-	Failed	-	-	-	-	-	-		Not
A-7-6(20)	13	65.3	79.0	120	62.00+	6.0	2.6	4.3	5	ND	24.1	NL	Recommended
	15	-	-	-	36.00+	-	-	-	-	-	-		
Perrin A	11	-	-	-	66.59+	-	-	-	-	-	-		Not
A-7-6(20)	13	89.9	140.7	156	58.65+	8.7	7.5	8.1	5	ND	24.5	ND	Recommended
	15	-	-	-	32.48+	-	-	-	-	-	-		

[a] + sign indicates that the Wt loss is more than the values as shown.

[b] The strength specimens ruptured and fell apart during intermediate F-T cycles. Therefore, average unconfined compressive strength of the specimens at the end of 12 F-T cycles was assumed to be zero.

[c] Not determined, because the measurements were not accurate at intermediate cycles due to soil-cement loss of the specimen.

criteria of maximum allowable weight losses after 12 freeze-thaw cycles, used for evaluating soil-cement mixtures, are given below:

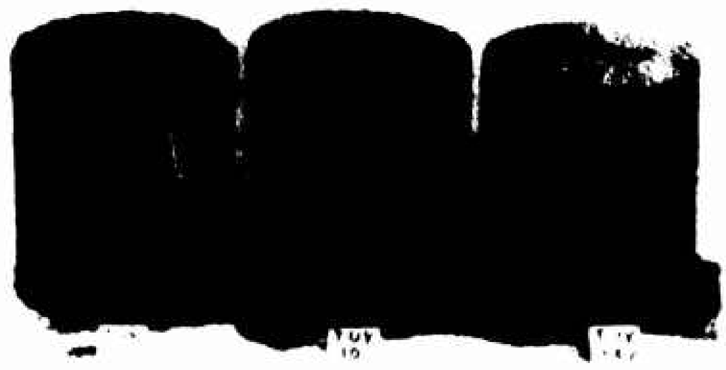
AASHO groups A-1, A-2-4, A-2-5 and A-3, not over 14 percent;

AASHO groups A-2-6, A-2-7, A-4 and A-5, not over 10 percent;

AASHO groups A-6 and A-7, not over 7 percent.

The high residual strengths and allowable weight losses (Table 10) indicated that Tuy Hoa, Altus subbase and Dyess mixtures were durable against freeze-thaw action. The specimens of these mixtures molded at their estimated cement contents also exhibited relatively high resistance to freezing and thawing as measured by their indices of resistance to freezing and thawing, R_f, defined as the strength after 12 freeze-thaw cycles divided by the 28-day strength. The photographs of brush specimens after 12 cycles of freezing and thawing (Figures 71 and 72) offer qualitative evidence that these specimens were durable.

Altus subgrade, Houma, Tyler, Perrin B and Perrin A specimens displayed very low residual strengths, high weight losses and very little or no resistance to freezing and thawing (as measured by R_f), indicating that the mixtures were not durable. These specimens, as shown in Figures 73 and 74, showed considerable weight losses by brushing after freeze-thaw cycles. It is probable that Tyler, Perrin B and Perrin A soils were not sufficiently hardened after 7-day curing due to the presence of sulfate and/or organics in the soils. Although Altus subgrade specimens exhibited reasonably high 7-day strengths, the high sulfate content (1.5 percent) probably caused deterioration in strength at a rapid rate after the specimens were subjected to water during the freeze-thaw cycles. It is suspected that the high organic content of the Houma soil interfered with its stabilization.

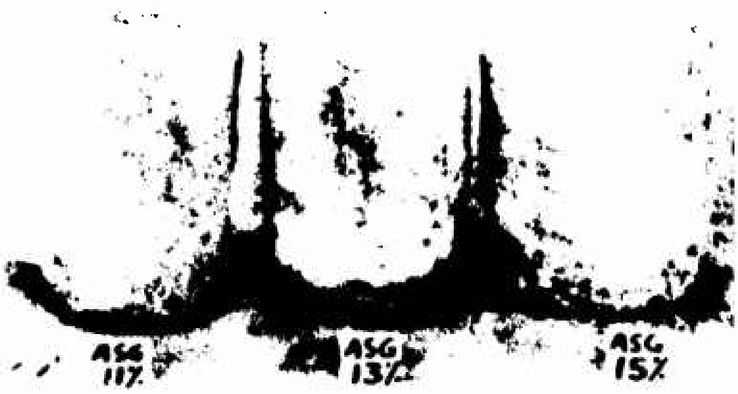


TUY HOA—12 FREEZE-THAW CYCLES

ALTUS SUBBASE— 12 FREEZE-THAW CYCLES

Figure 71 - Cement-Treated Tuy Hoa and Altus Subbase Specimens After Brushing. Numbers Indicate Percent Cement

DYESS — 12 FREEZE-THAW CYCLES



ALTUS SUBGRADE — 7 FREEZE-THAW CYCLES

Figure 72 - Cement-Treated Dyess and Altus Subgrade Specimens After Brushing. Numbers Indicate Percent Cement.

HOUMA—12 FREEZE-THAW CYCLES

TYLER—3 FREEZE-THAW CYCLES

Figure 73 - Cement-Treated Houma and Tyler Specimens After Brushing. Numbers Indicate Percent Cement

PERRIN B — 5 FREEZE-THAW CYCLES

PERRIN A — 5 FREEZE-THAW CYCLES

Figure 74 - Cement-Treated Perrin B and Perrin A Specimens After Brushing. Numbers Indicate Percent Cement

b. Unit length change.

The effect of freeze-thaw cycles on the unit length changes of the soil-cement specimens is shown in Figures 75 through 82. Tuy Hoa, Altus subbase and Dyess specimens had very high resistance to unit length changes (Figures 75, 76, 77). Altus subgrade, Houma, Perrin B and Perrin A specimens exhibited larger unit length changes during freezing and thawing (Figures 78 through 81). Tyler specimens collapsed after 2 cycles of freezing and thawing (Figure 82).

c. Moisture gain.

Tuy Hoa, Altus subbase and Dyess specimens showed very little or no moisture content increases above their molding moisture contents (Figures 75 through 77). The less durable mixtures, Altus subgrade, Houma, Perrin B and Perrin A, exhibited large moisture content increases. Measurements were discontinued on Perrin B and Perrin A specimens after 4 and 6 freeze-thaw cycles, respectively, because of excessive loss of soil, and after 2 cycles on the Tyler specimens.

d. Summary - freeze-thaw test.

Weight loss, residual strength, unit length change and moisture gain after freeze-thaw cycles were all effective methods of evaluating freeze-thaw durability of soil-cement mixtures. The durable soil-cement mixtures, Tuy Hoa, Altus subbase and Dyess, displayed high residual strengths, small weight losses, small unit length changes and little or no moisture increases above their molding moisture contents. Altus subgrade, Houma, Tyler, Perrin B and Perrin A specimens possessed poor freeze-thaw durability properties based on the above criteria.

6. Results of Wet-Dry Test

The methods used for evaluating wet-dry (W-D) durability of soil-cement mixtures were weight loss, residual strength, unit length change and moisture gain.

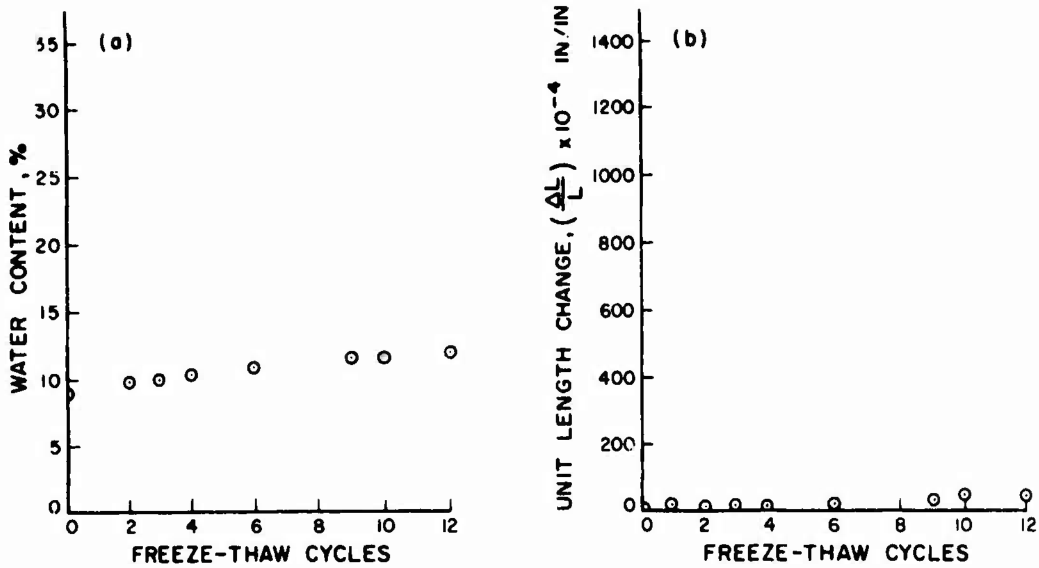


Figure 75 - Influence of Freeze-Thaw Cycles on Water Content and Unit Length Change. Tuy Hoa Soil + 10 percent Cement

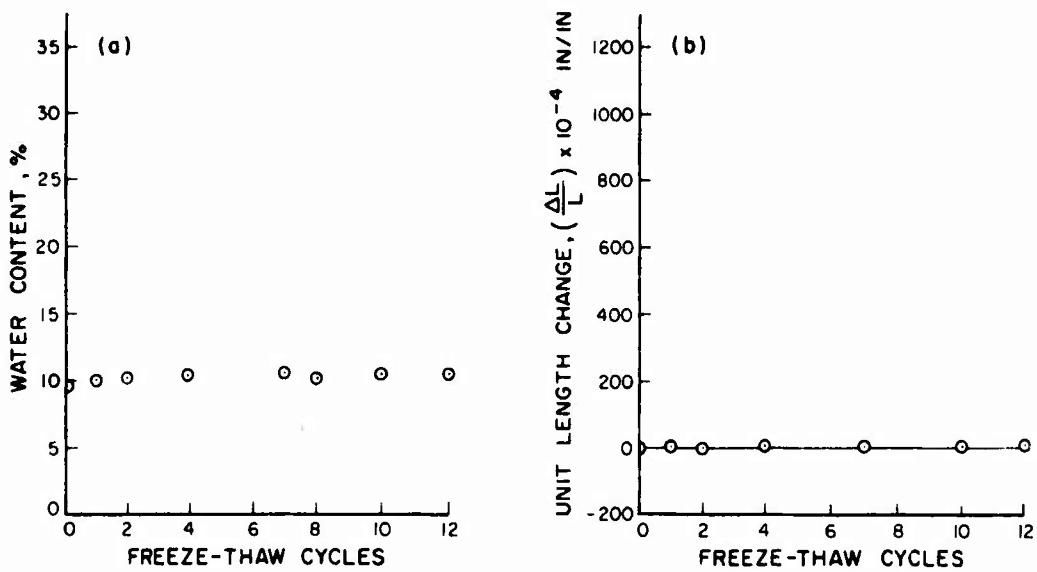


Figure 76 - Influence of Freeze-Thaw Cycles on Water Content and Unit Length Change. Altus Subbase Soil + 7 percent Cement

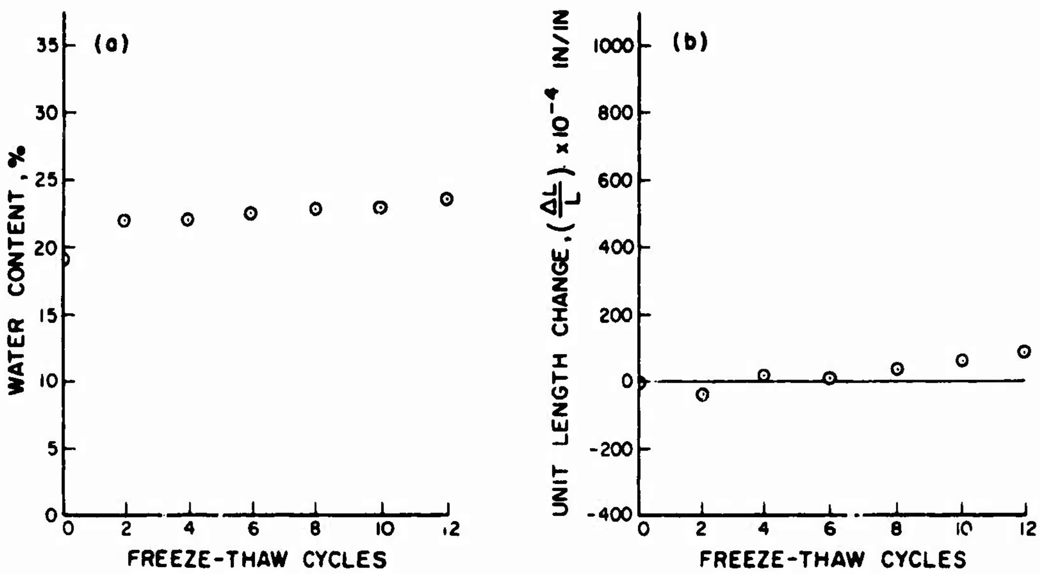


Figure 77 - Influence of Freeze-Thaw Cycles on Water Content and Length Change. Dyess + 12 percent Cement

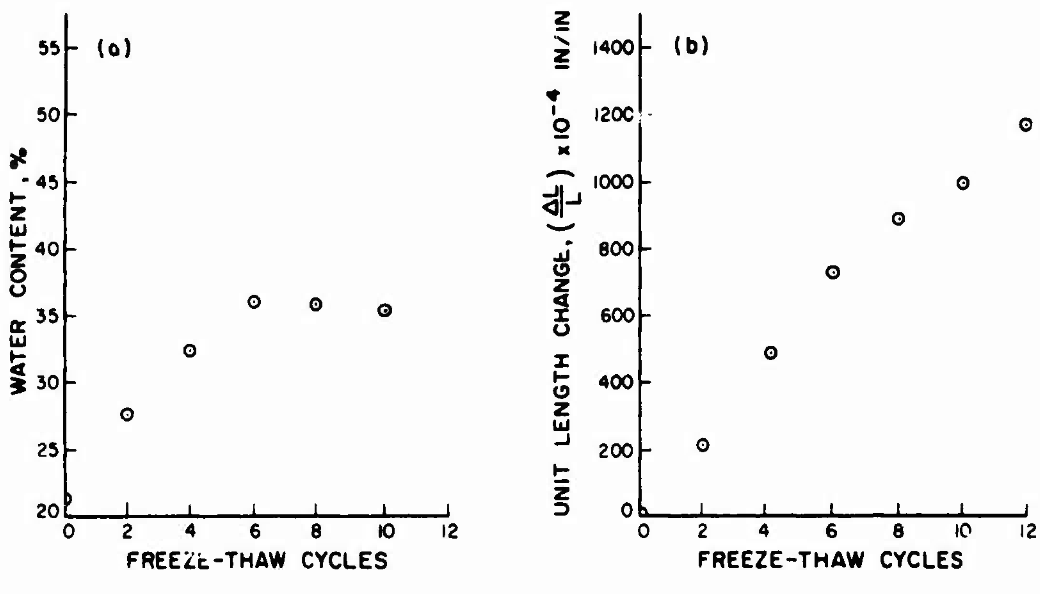


Figure 78 - Influence of Freeze-Thaw Cycles on Water Content and Unit Length Change. Altus Subgrade + 13 percent Cement

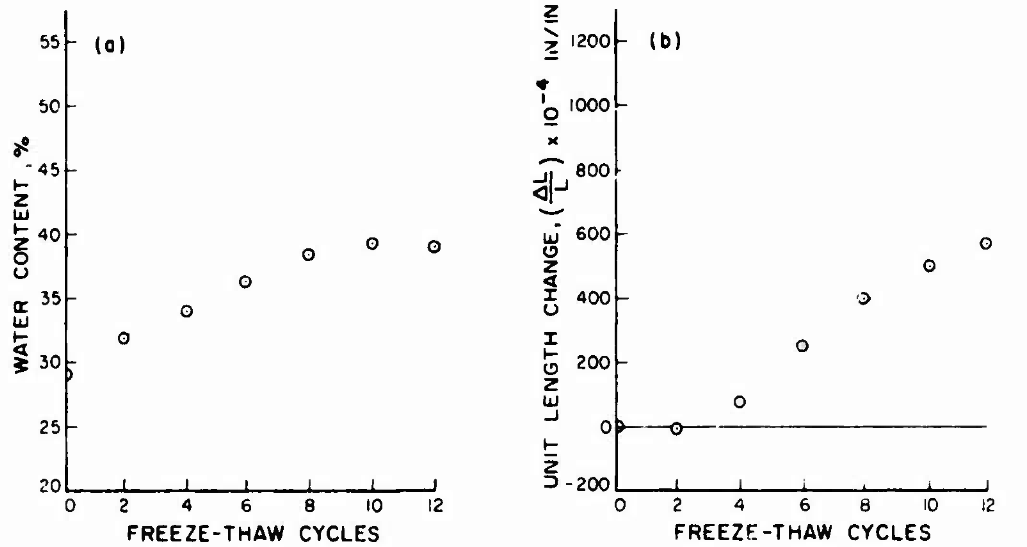


Figure 79 - Influence of Freeze-Thaw Cycles on Water Content and Unit Length Change. Houma Soil + 13 percent Cement

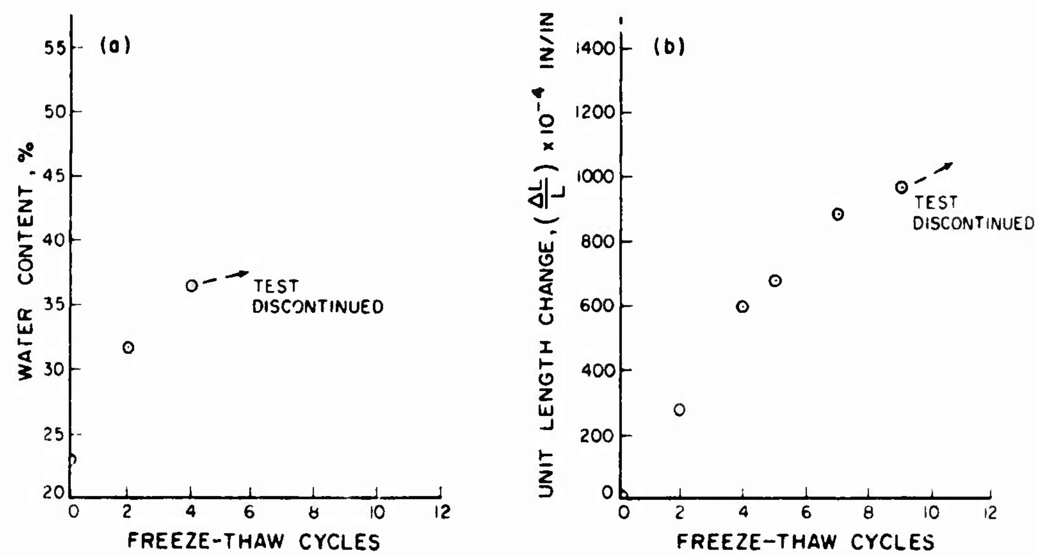


Figure 80 - Influence of Freeze-Thaw Cycles on Water Content and Unit Length Change. Perrin B Soil + 13 percent Cement

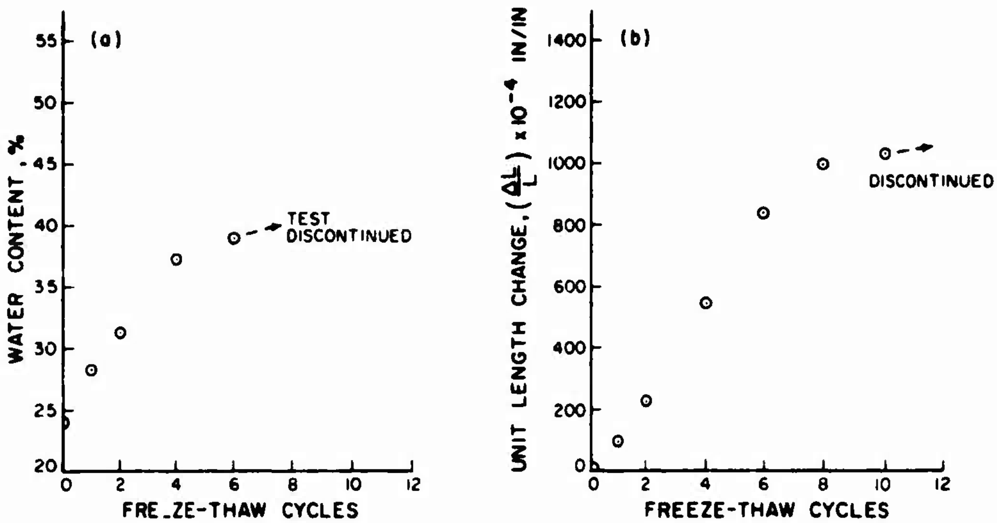


Figure 81 - Influence of Freeze-Thaw Cycles on Water Content and Unit Length Change. Perrin A Soil + 13 percent Cement

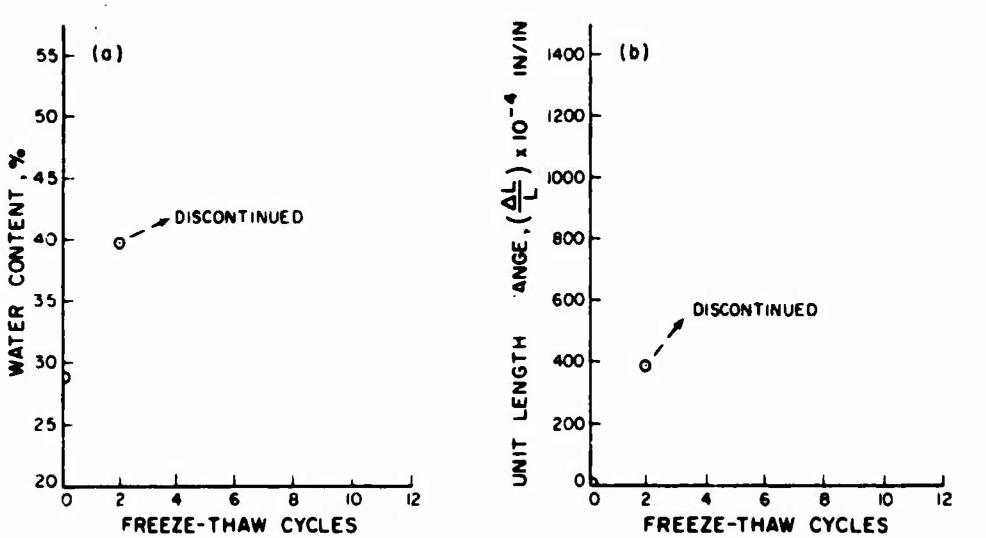


Figure 82 - Influence of Freeze-Thaw Cycles on Water Content and Unit Length Change. Tyler Soil + 13 percent Cement

a. Weight loss and residual strength.

Figure 70 shows residual strengths after 12 cycles of wetting and drying for specimens molded at the estimated cement content. Table 11 shows residual strengths and weight losses for all of the soil-cement mixtures. The PCA criteria of maximum allowable weight losses used for evaluating wet-dry durability of specimens are the same as those for freeze-thaw specimens.

The small percentage of weight loss, high residual strength and high index of resistance to wetting and drying (R_w) observed for Tuy Hoa, Altus subbase, Dyess and Altus subgrade mixtures after 12 cycles of wetting and drying indicated that these mixtures were very durable (Table 11). Figures 83 and 84 also show qualitatively that these mixtures were durable after 12 wet-dry cycles. The compressive strengths after 12 wet-dry cycles were well above their respective 28-day strengths (Table 11) and were also much higher than the strengths after 12 freeze-thaw cycles. The high temperature (160°F) during the drying phase of the wet-dry test may have increased the cementing reaction, leading to the high strength gains.

Houma, Tyler, Perrin B and Perrin A specimens did not meet the weight loss or unconfined compressive strength criteria (Table 11 and Figures 85 and 86), indicating that these soils were not durable. Organic matter and/or sulfate were probably responsible for the poor durability.

b. Unit length change.

The unit length change was an effective method of measuring the durability of soil-cement mixtures. The plots for unit length change after the drying phase and the wetting phase at each wet-dry cycle are shown in Figures 87 through 94.

Tuy Hoa specimens experienced practically no unit length change during

TABLE 11

SUMMARY OF WET-DRY TEST RESULTS OF SOIL-CEMENT MIXTURES

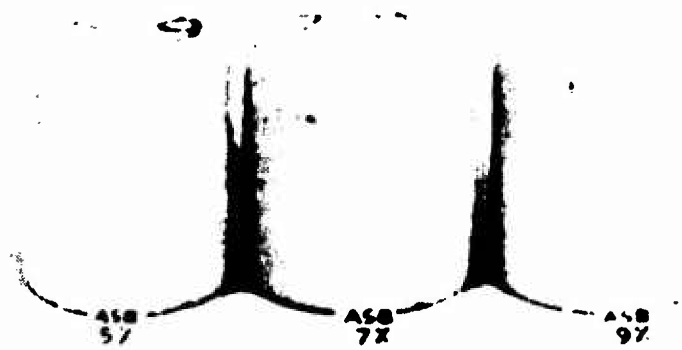
	Percent Cement	7-Day Strength q_7, psi	28-Day Strength q, psi	$\frac{q}{q_7}$ %	Wt Loss Percent After 12 W-D Cycles	Unconfined Compressive Strength After 12 W-D Cycles, q_f, psi: Specimen 1	Specimen 2	Average	R_v %	Unit Length Change After 12 W-D Cycles in/in ($\times 10^{-4}$)	Water Content of Molded Specimens Percent	Water Content of Specimens After 12 W-D Cycles Percent	Recommended Percent Cement
Tuy Hoa	6	166.3	215.5	129	7.33	287.3			133	-	-	-	
A-1b	10	492.6	693.9	140	2.31	983.6	926.3	955.0	137	7	9.3	2.2	10
	13	545.9	961.3	176	0.94	1333.7	1734.0	1533.9	159	-	-	-	
Altus	5	279.5	550.7	197	10.08	1032.9	1149.1	1091.0	198	-	-	-	
Subbase	7	406.4	875.4	215	4.94	1370.3	1704.5	1537.4	175	-42	10.4	2.7	5
A-2-4	9	493.4	977.2	198	2.65	1567.7	1265.3	1416.5	144	-	-	-	
Dyess	10	-	-	-	4.25	-	-	-	-	-	-	-	
A-7-6(12)	12	506.9	823.5	162	2.07	1363.0	1479.2	1421.1	172	-66	19.9	6.2	10
	14	-	-	-	1.35	-	-	-	-	-	-	-	
Altus	11	-	-	-	4.58	-	-	-	-	-	-	-	
Subgrade	13	383.2	507.6	132	1.78	930.1	1119.5	1024.8	201	-9	22.2	7.7	11
A-7-6(12)	15	-	-	-	0.03	-	-	-	-	-	-	-	
Tyler	11	-	-	-	Failed	-		-	-		-		Not
A-7-5(15)	13	51.7	66.9	129	55.70	39.3	63.3	51.3	76	ND[a]	30.3	ND	Recommended for
	15	-	-	-	50.10	-	-	-	-		-		Cement Stabilization
Houma	11	-	-	-	24.85	-	-	-	-		-		Not
A-7-6(20)	13	354.5	313.1	123	20.64	271.1	-	271.1	86	ND	29.5	ND	Recommended for
	15	-	-	-	15.67	-	-	-	-		-		Cement Stabilization
Perrin B	11	-	-	-	Failed						-		Not
A-7-6(20)	13	65.3	79.0	120	Failed	0[b]	0[b]	0	0	ND	24.1	ND	Recommended for
	15	-	-	-	Failed						-		Cement Stabilization
Perrin A	11	-[a]	-	-	Failed						-		Not
A-7-6(20)	13	89.9	140.7	156	Failed	0[c]	0[c]	0	0	ND	24.5	ND	Recommended for
	15	-	-	-	Failed						-		Cement Stabilization

[a] Not determined, because the measurements were not accurate at intermediate cycles due to soil-cement loss of specimens.

[b] After 2 W-D cycles the strength specimens collapsed. Therefore the strength of the specimen after 12 W-D cycles was assumed to be zero.

[c] After e W-D cycles the strength specimen collapsed. Therefore the strength after 12 W-D cycles was assumed to be zero.

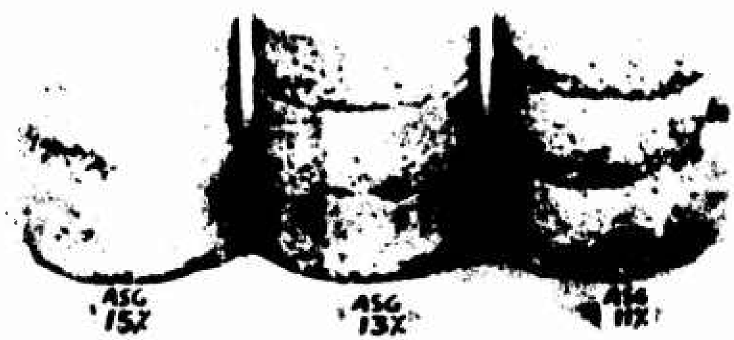
TUY HOA — 12 WET-DRY CYCLES



ALTUS SUBBASE — 12 WET-DRY CYCLES

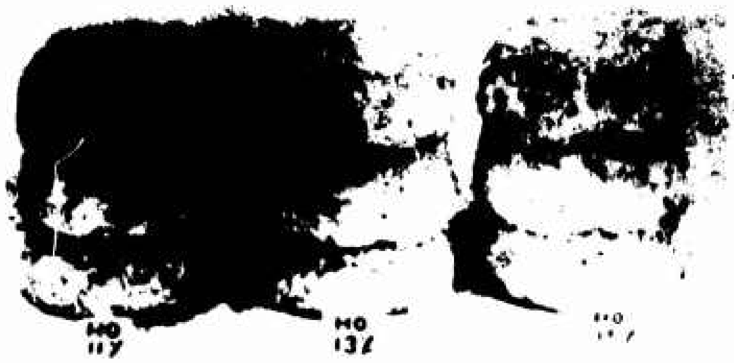
Figure 83 - Cement-Treated Tuy Hoa and Altus Subbase Specimens After Brushing. Numbers Indicate Percent Cement

DYESS — 12 WET-DRY CYCLES



ALTUS SUBGRADE — 12 WET-DRY CYCLES

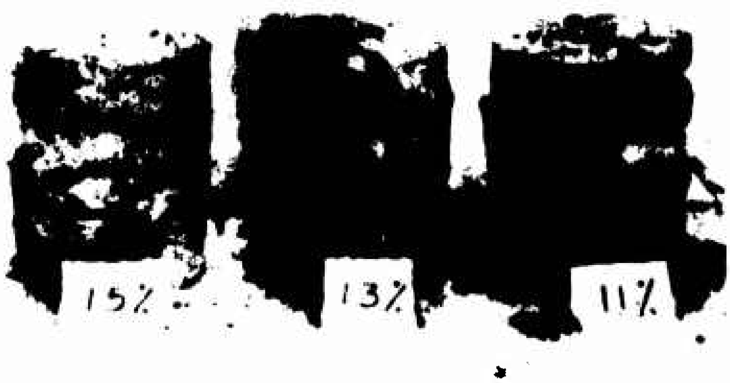
Figure 84 - Cement-Treated Dyess and Altus Subgrade Specimens After Brushing. Numbers Indicate Percent Cement



HOUMA — 12 WET-DRY CYCLES

TYLER — 12 WET-DRY CYCLES

Figure 85 - Cement-Treated Houma and Tyler Specimens After Brushing. Numbers Indicate Percent Cement



PERRIN B — 2 WET-DRY CYCLES

PERRIN A — 3 WET-DRY CYCLES

Figure 86 - Cement-Treated Perrin B and Perrin A Specimens After Brushing Numbers Indicate Percent Cement

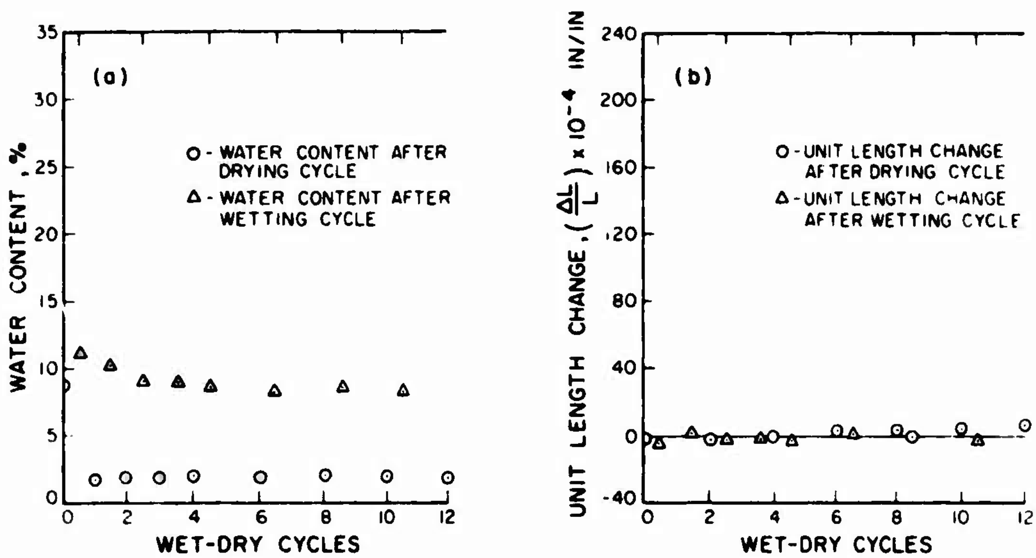


Figure 87 - Influence of Wetting and Drying on Water Content and Unit Length Change. Tuy Hoa Soil + 10 percent Cement

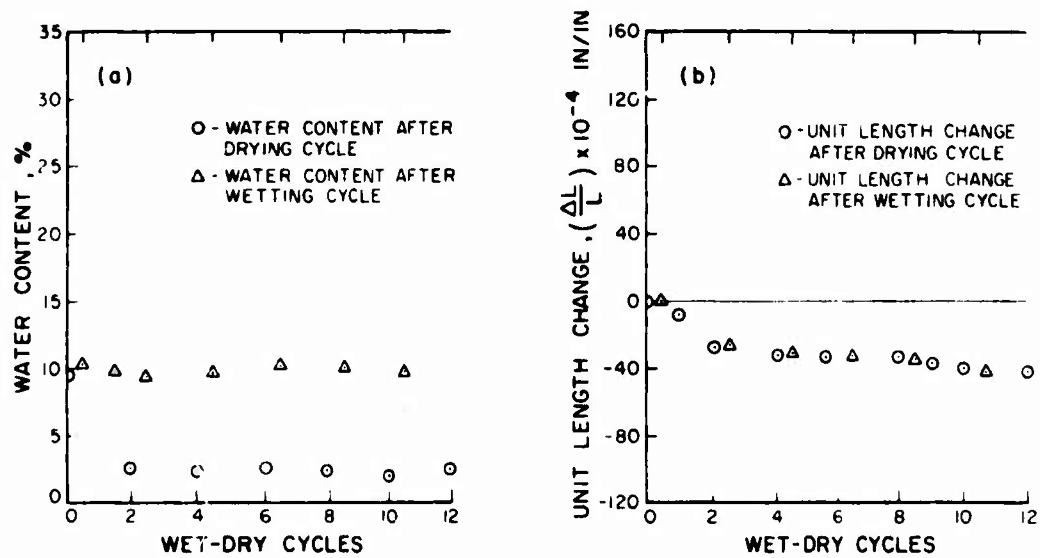


Figure 88 - Influence of Wetting and Drying on Water Content and Unit Length Change. Altus Subbase Soil + 7 percent Cement

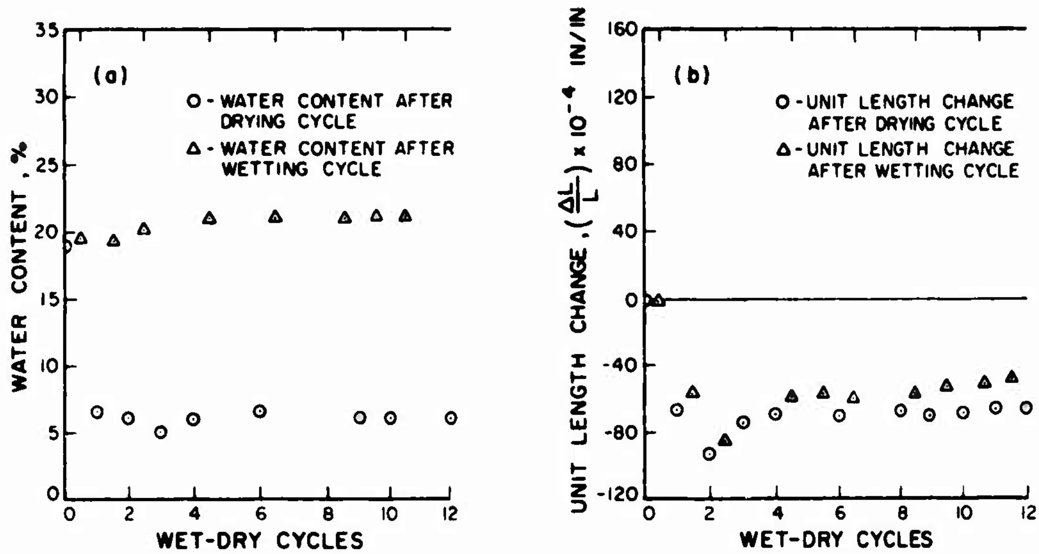


Figure 89 - Influence of Wetting and Drying on Water Content and Unit Length Change. Dyess Soil + 12 percent Cement

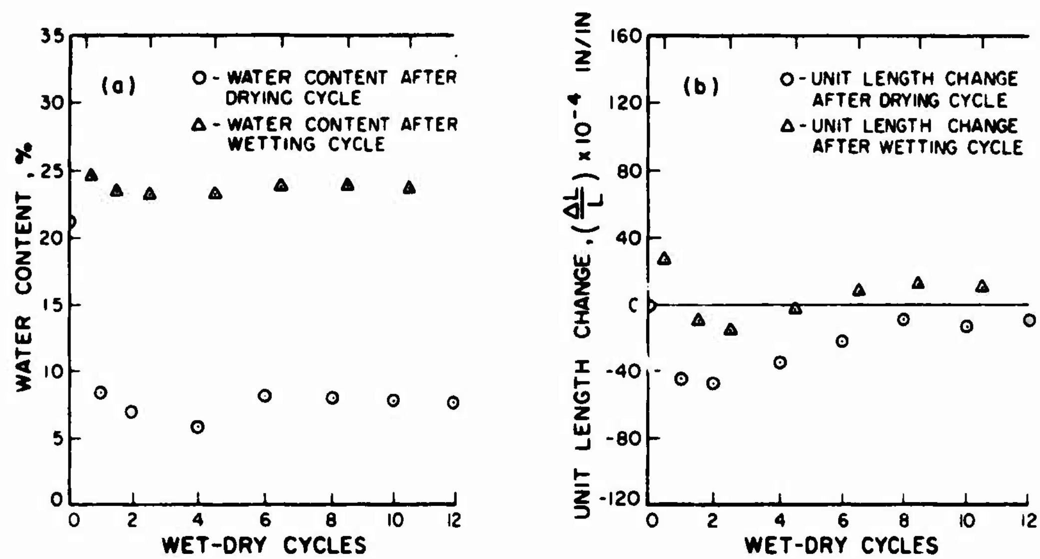


Figure 90 - Influence of Wetting and Drying on Water Content and Unit Length Change. Altus Subgrade Soil + 13 percent Cement

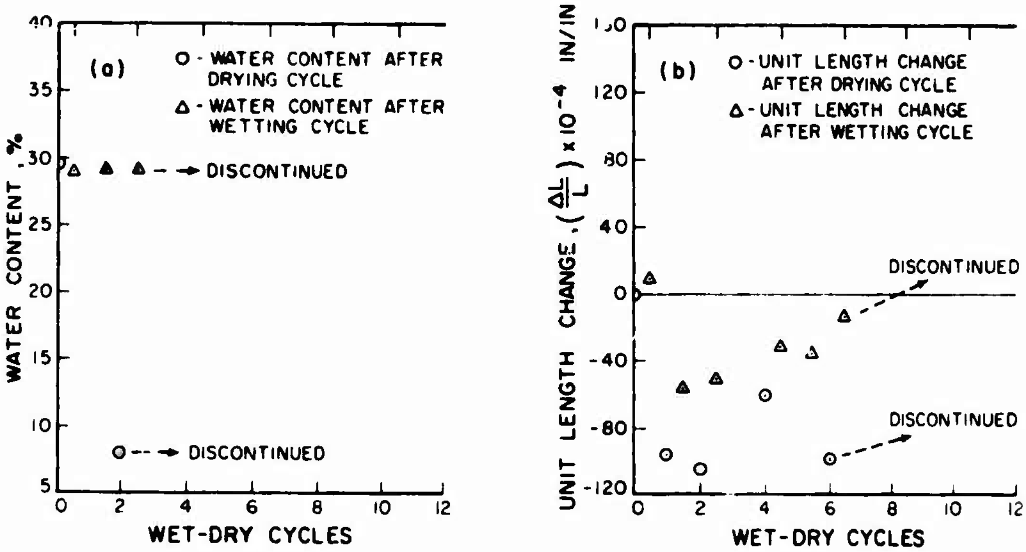


Figure 91 - Influence of Wetting and Drying on Water Content and Unit Length Change. Houma Soil + 13 percent Cement

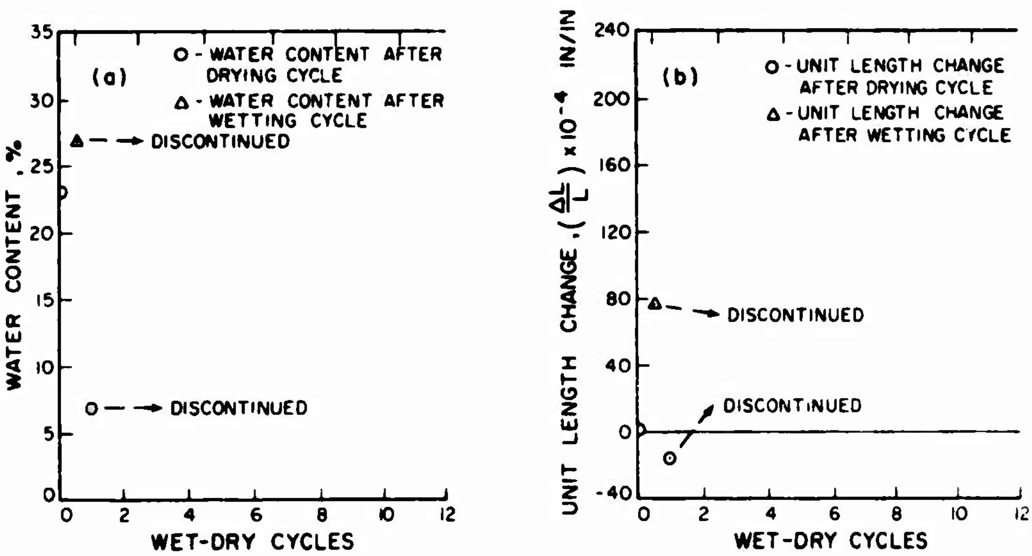


Figure 92 - Influence of Wetting and Drying on Water Content and Unit Length Change. Perrin B Soil + 13 percent Cement

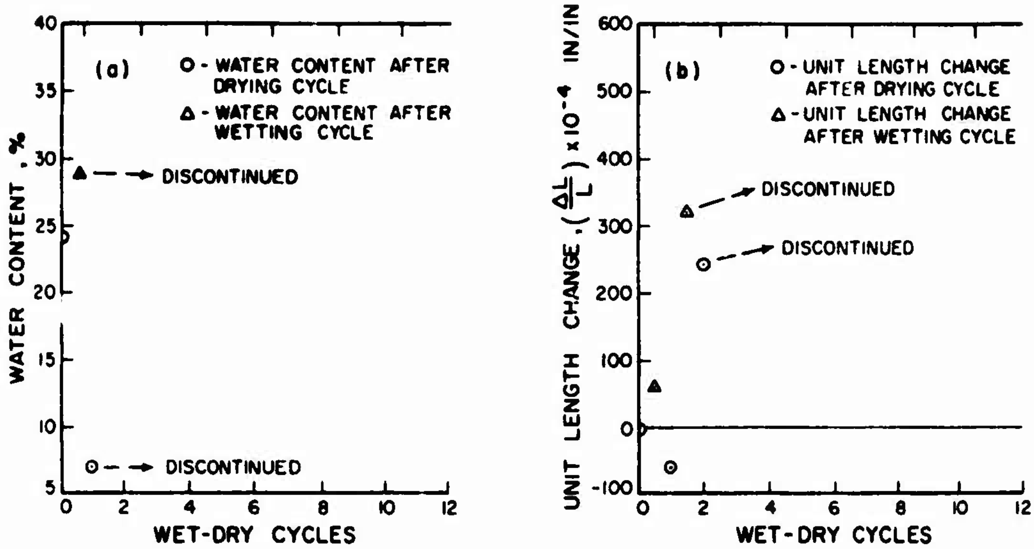


Figure 93 - Influence of Wetting and Drying on Water Content and Unit Length Change. Perrin A Soil + 13 percent Cement

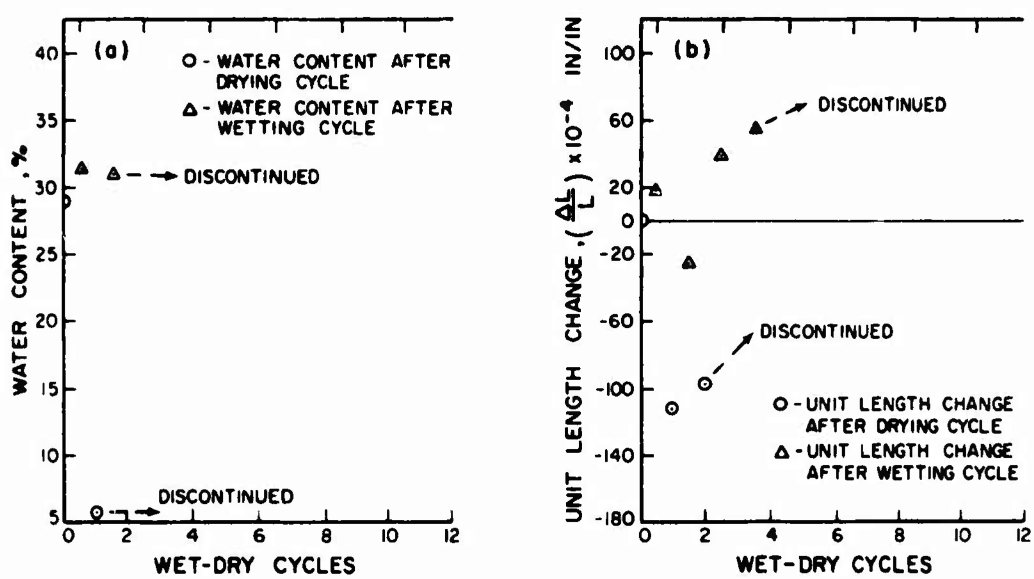


Figure 94 - Influence of Wetting and Drying on Water Content and Unit Length Change. Tyler Soil + 13 percent Cement

wet-dry cycles and showed very good durability. Altus subbase, Dyess and Altus subgrade soils also displayed substantial resistance to unit length changes.

Houma specimens initially decreased in length, and then started increasing in length. The mixtures with Perrin B and Perrin A soils cracked and then collapsed after being subjected to 2 and 3 wet-dry cycles, respectively. Accurate length measurements could not be obtained for Tyler mixture specimens after 4 wet-dry cycles due to the uneven specimen surfaces.

c. Moisture gain.

Figures 87 through 94 show the effects of wet-dry cycles on moisture changes in the soil-cement specimens. Tuy Hoa, Altus subbase, Dyess and Altus subgrade mixtures exhibited little or no moisture content increases after each wetting cycle. The moisture contents of Houma, Perrin B, Perrin A and Tyler specimens could not be determined after 1 to 2 cycles of wetting and drying owing to damage of the specimens.

d. Summary - wet-dry test.

Cement-stabilized Tuy Hoa, Altus subbase, Dyess and Altus subgrade soils exhibited low weight losses, high residual strengths and negligible unit length changes after 12 wet-dry cycles. They also showed little or no moisture content increases above their molding moisture contents after the wetting cycles. The less durable mixtures, Houma, Perrin B, Perrin A and Tyler mixtures, exhibited little or no residual strengths and considerable weight losses. These mixtures were also unsuitable for determining unit length changes or moisture changes, because specimens were either completely damaged or gave inaccurate weight measurements due to losses at the intermediate wet-dry cycles. Thus, it appears that low weight loss, high residual strength, low unit length change and low moisture content increases were indicative of good wet-dry durability of soil-cement mixtures.

7. Long-Term Immersion Test

Long-term immersed strengths (unconfined compressive strength after 7 days normal curing and 21 days immersion in water) of the different soil-cement specimens are shown in Table 12 and Figure 70. The soil-cement specimens of Tuy Hoa, Altus subbase, Dyess and Altus subgrade soils possessed good immersion durability properties: these specimens at their estimated cement contents exhibited high immersed strengths and high indices of resistance to long-term immersion, R_{ii}. Houma had a reasonably high immersed strength and index of resistance to immersion, indicating that the mixture was reasonably durable against long-term immersion. Although organic matter in the Houma soil was probably responsible for the relatively low strength after 7-day normal curing, the strength of the cured specimen actually improved during the subsequent 21-day immersion period. The low immersed strengths observed for Tyler, Perrin B and Perrin A mixtures indicated that these soil-cement mixtures were not durable. These three soils were not successfully stabilized with cement.

8. Effect of Sulfate on Cement Stabilization

To study the effect of sulfate on cement stabilization, the organic soils Tuy Hoa, Dyess and Houma were not considered. Using the data given in Table 13, Figures 95 and 96 were prepared which show 7-day strengths and durabilities plotted versus sulfate contents. If the Altus subgrade soil is not considered for the moment, these figures show that a sulfate content of 1 percent was adequate to cause low 7-day strengths and low durabilities. It is somewhat puzzling that the Altus subgrade soil, containing 1.5 percent sulfate, exhibited such high 7-day strength and high resistance to long-term immersion and wet-dry cycles. However, previous research has indicated that the degree of disinte-

TABLE 12

SUMMARY OF IMMERSION TEST RESULTS OF SOIL-CEMENT MIXTURES

Soil & Class	Percent Cement	7-Day Strength, q_7 psi	28-Day Strength, psi	q/q_7 %	Long Term Immersed Strength, q_{ii}, psi			R_{ii} %	Recommended Percent Cement
					Specimen				
					1	2	Average		
Tuy Hoa	6	156.3	215.5	129	201.2	226.6	213.9	99	
	10	492.6	693.9	140	518.8	604.8	561.8	80	10
	13	545.9	961.3	176	795.8	802.1	799.0	83	
Altus Subbase A-2-4	5	279.5	550.7	197	447.2	432.9	440.1	79	5
	7	406.4	875.4	215	790.2	623.9	707.1	80	
	9	493.4	977.2	198	865.8	-	865.8	88	
Dyess A-7-6(12)	10	-	-	-	-	-	-	-	
	12	506.9	823.5	162	686.6	669.1	677.8	82	10
	14	-	-	-	-	-	-	-	
Altus Subgrade A-7-6(12)	11	-	-	-	-	-	-	-	
	13	383.2	507.6	132	543.4	694.6	619.0	121	11
	15	-	-	-	-	-	-	-	
Tyler A-7-5(15)	11	-	-	-	-	-	-	-	Not Recommended
	13	51.7	66.9	129	25.7	28.9	27.3	40	
	15	-	-	-	-	-	-	-	
Houma A-7-6(20)	11	-	-	-	-	-	-	-	
	13	254.5	313.1	123	332.6	327.9	330.3	105	> 15
	15	-	-	-	-	-	-	-	
Perrin B A-7-6(20)	11	-	-	-	-	-	-	-	Not Recommended
	13	65.3	79.0	120	45.6	44.0	44.8	56	
	15	-	-	-	-	-	-	-	
Perrin A A-7-6(20)	11	-	-	-	-	-	-	-	Not Recommended
	13	89.9	140.7	156	100.9	100.5	100.7	71	
	15	-	-	-	-	-	-	-	

TABLE 13

SEVEN-DAY STRENGTHS AND DURABILITIES OF CEMENT-STABILIZED

SULFATE AND ORGANIC SOILS AT THEIR ESTIMATED CEMENT CONTENTS

Soil	Percent Sulfate	Percent Organic Matter	Percent Cement	Unconfined Compressive Strength, psi			
				After 7-Day Normal Curing	After Long-Term Immersion	After 12 W-D Cycles	After 12 F-T Cycles
Tuy Hoa	0.00	3.5	10	492.6	561.8	955.0	559.5
Altus Subbase	0.09	0.15	7	406.4	707.1	1537.4	473.1
Dyess	0.00	1.17	12	506.9	677.8	1421.1	387.5
Altus Subgrade	1.52	0.07	13	383.2	619.0	1024.8	27.5
Tyler	3.50	3.0	13	51.7	27.3	51.3	0.00
Houma	0.00	1.43	13	254.5	330.3	271.1	68.9
Perrin B	7.64	0.13	13	65.3	44.8	0.00	4.3
Perrin A	1.00	0.00	13	89.9	100.7	0.00	8.1

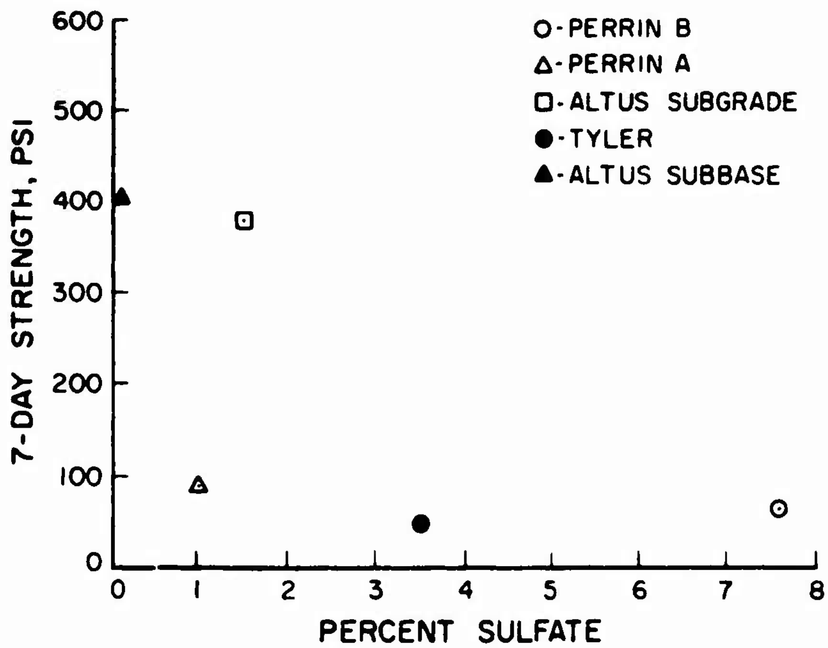


Figure 95 - Effect of Sulfate on 7-Day Strengths of Soil-Cement Mixtures

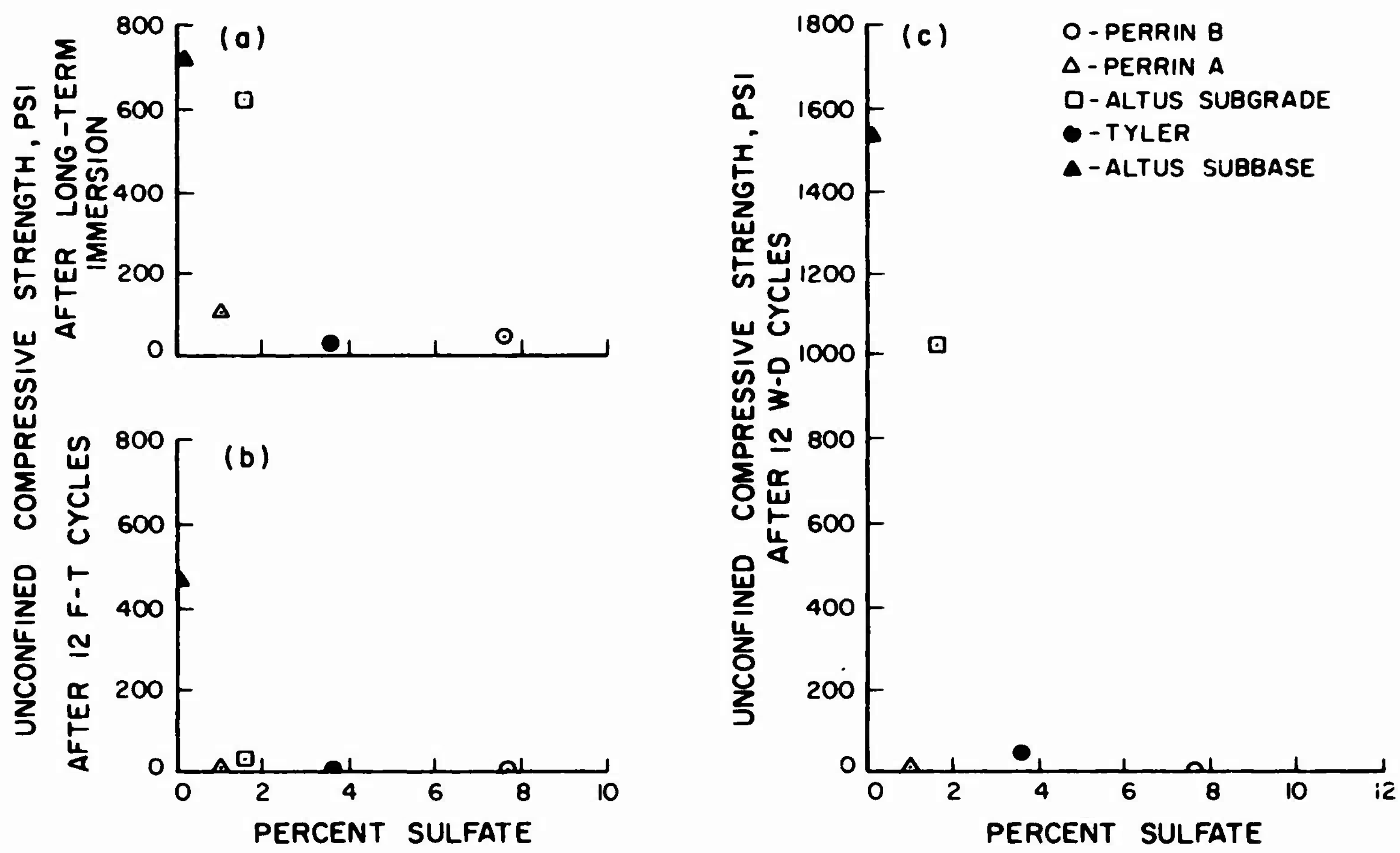


Figure 96 - Effect of Sulfate on Immersion, Freeze-Thaw and Wet-Dry Durability of Soil-Cement Mixtures

gration of soil-cement resulting from sulfate ions is proportional to the amount of clay present in the soil; this is attributed to a reaction that occurs between clay and sulfate ions in the presence of lime liberated from the cement and excess water (reference 16). On this basis, the Altus subgrade soil, with a relatively small amount of -2μ clay compared to the more plastic Perrin A and Perrin B soils, should be less affected by sulfate than the Perrin A and Perrin B soils. Better mixing of the cement into the medium plastic Altus subgrade may have helped it outperform the highly plastic Perrin A and Perrin B soils, but this did not seem to help the medium plastic Tyler soil. Its high sulfate content offset any beneficial effects due to good mixing.

Thus, it appears that the allowable sulfate content depends on the type of soil and kind of durability test. About 1 percent sulfate renders a soil incapable of withstanding freeze-thaw action, but if the soil is of medium plasticity, its sulfate content may be as high as 1.5 percent before it succumbs to long-term immersion and wet-dry cycles. This is based on a very limited amount of data, and it tends to reinforce the restriction of 0.90 percent allowable sulfate content originally suggested in SSIS.

. Effect of Organics on Cement Stabilization

The high sulfate-bearing soils, i.e., Perrin B, Perrin A and Altus subgrade, were not included when analyzing the effect of organic matter on cement stabilization of soils, although the Tyler soil was included. The 7-day strengths and durabilities of soil-cement mixtures plotted versus percent organic matter are shown in Figures 97 and 98. Data are also shown in Table 13. Dyess and Altus subbase specimens, with 1.17 and 0.15 percent organics, respectively, exhibited high 7-day strengths and good durabilities. The high plastic Houma soil, with 1.4 percent organics, did not have substantial strength or

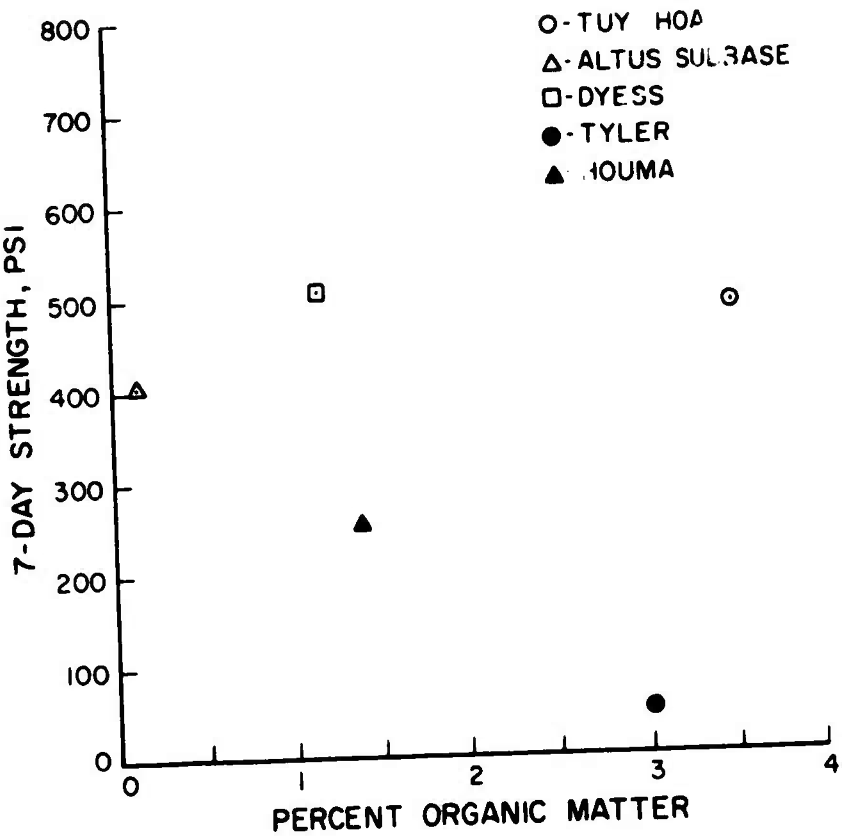


Figure 97 - Effect of Organic Matter on 7-Day Strengths of Soil-Cement Mixtures

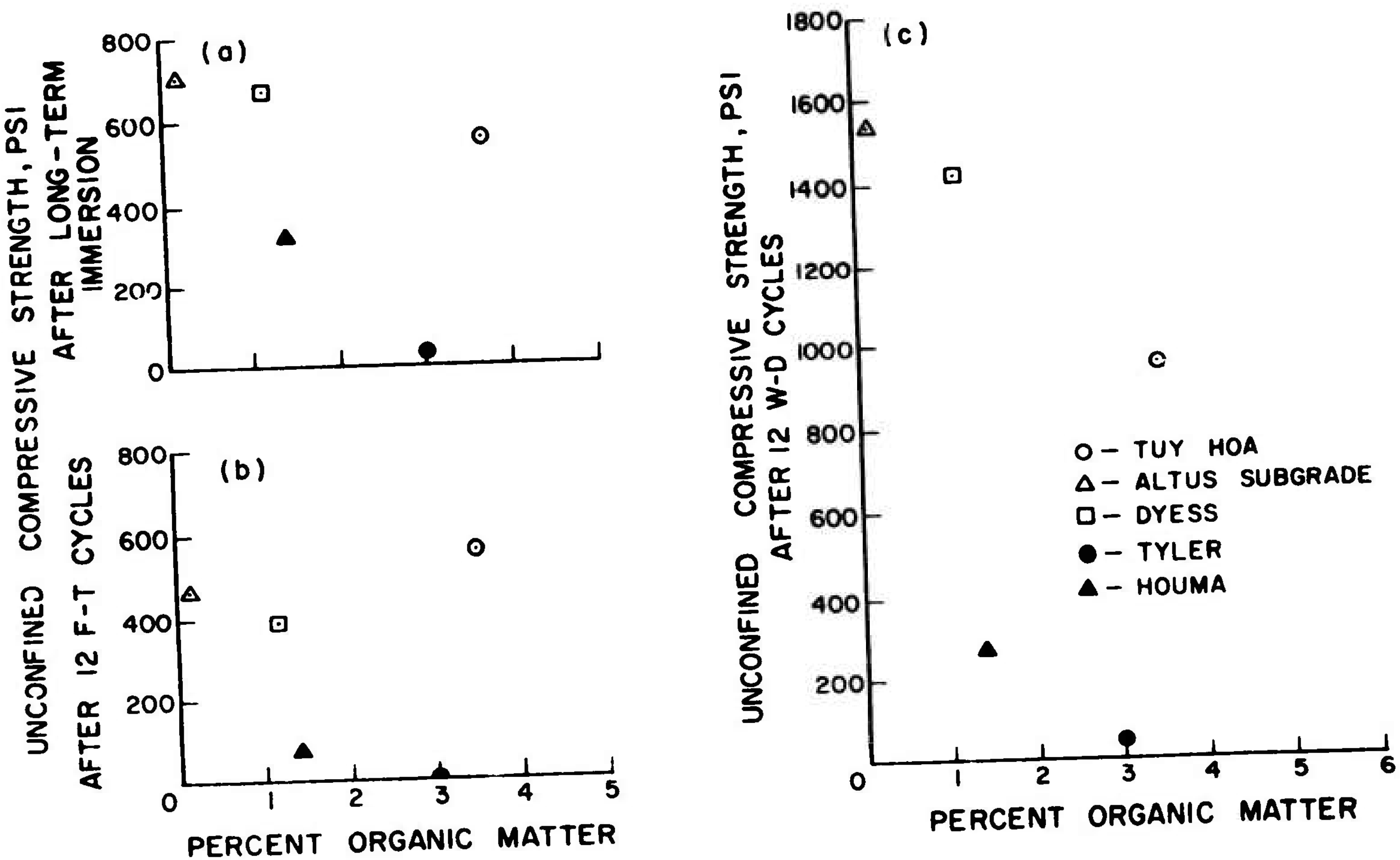


Figure 98 - Effect of Organic Matter on Immersion, Freeze-Thaw and Wet-Dry Durability of Soil-Cement Mixtures

durability. But the cohesionless Tuy Hoa soil containing 3.5 percent organic matter was well stabilized with cement and possessed good durability. Although the amount of organic matter in the Houma soil was substantially less than in the Tuy Hoa soil, the organic matter in the former appeared to be more active and intefered with cement stabilization. The Tyler soil reacted poorly with cement, but it is not certain whether this was due to organics or sulfates.

Therefore, based on these limited data, it is not possible to place a value on the maximum percentage of organic matter for all soils that will not interfere with cement stabilization. However, the test results show that soils containing less than about 1.4 percent organic matter were well stabilized with cement and durable in different kinds of durability tests.

10. Use of 7-Day Strength for Prediction of 28-Day Strength and Durability

The long time involved in evaluating soil-cement mixtures by 28-day strengths and either freeze-thaw or wet-dry durability is a definite disadvantage for many construction projects, particularly for expedient situations demanded by the military. To determine whether the 7-day strength showed promise of predicting long-term behavior, several relationships were studied.

A plot of 7-day strength versus 28-day strength is shown in Figure 99. Linear regression analysis indicated that the relationship between the two strengths could be satisfactorily expressed as follows:

$$q = 1.694\ q_7$$

The correlation coefficient, R, was 0.9. Thus, on the basis of these limited data, it appears that the 28-day strength can be predicted with sufficient accuracy from the 7-day strength.

Freeze-thaw or wet-dry durability of soil-cement mixtures was measured

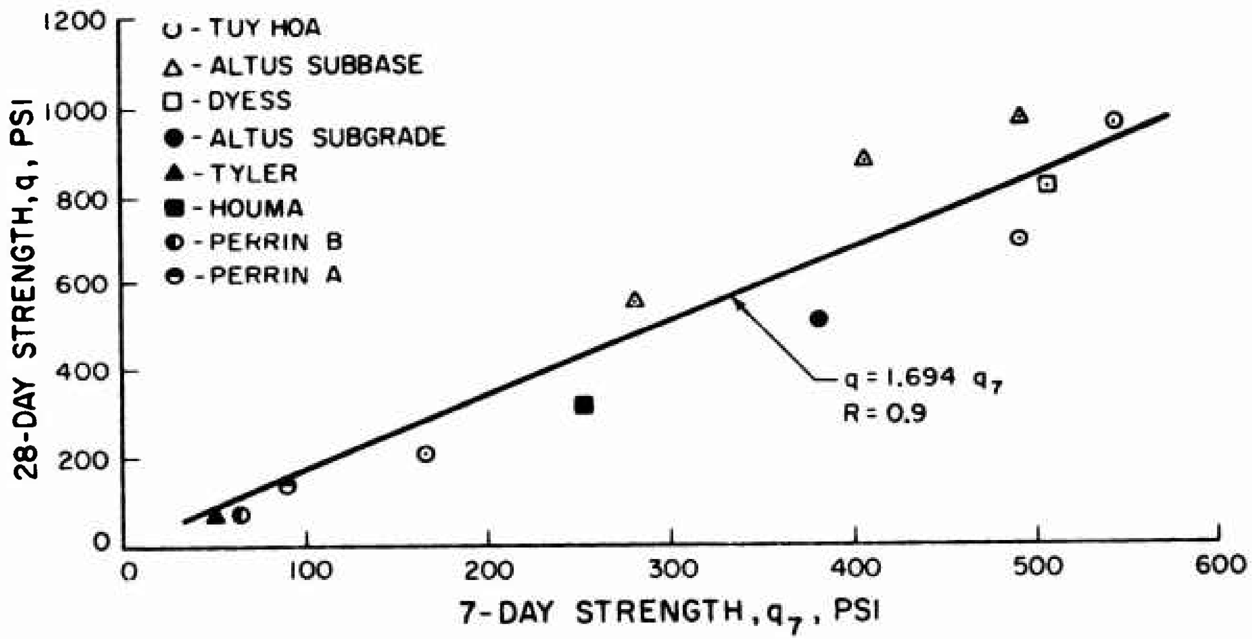


Figure 9? - Relation Between 7-Day and 28-Day Strengths for Soil-Cement Mixtures

adequately by weight loss and residual strength (unconfined compressive strength after 12 freeze-thaw or 12 wet-dry cycles). As previously indicated, the PCA criteria for weight loss in either freeze-thaw or wet-dry test are:

Groups A-1, A-2-4, A-2-5 and A-3, not over 14 percent;

Groups A-2-6, A-2-7, A-4 and A-5, not over 10 percent;

Groups A-6 and A-7, not over 7 percent.

Their experience has also shown that a soil-cement mixture whose compressive strength is approximately 300 psi or more after 7 days and is increasing, will pass the wet-dry and freeze-thaw tests.

Weight losses after freeze-thaw cycles are plotted versus the 7-day strengths in Figure 100 and similar plots are also shown for wet-dry cycles (Figure 101). Weight losses for the freeze-thaw specimens of Perrin B, Perrin A, Altus subgrade and Tyler far exceeded the PCA limits and these tests were discontinued at intermediate cycles. For plotting purposes, it was assumed that these materials suffered 100 percent weight loss after 12 freeze-thaw cycles. Figure 100 shows that the minimum 7-day strengths (excluding the Altus subgrade specimens) required to meet the allowable weight losses of 14 and 7 percent were approximately 300 and 350 psi, respectively. Figure 101 shows that the corresponding 7-day strengths for wet-dry tests would be approximately 240 and 310 psi for weight losses of 14 and 7 percent, respectively. Thus, it appears that an average minimum 7-day strength of 325 psi would be necessary for soil-cement specimens to meet freeze-thaw weight loss criteria, and 275 psi would be required for wet-dry tests. These data indicate that the PCA requirement of 300 psi is slightly unconservative for freeze-thaw tests and conservative for wet-dry tests.

A study was also made to determine whether any relationship exists between 7-day strength and residual strength after 12 freeze-thaw or 12 wet-dry cycles.

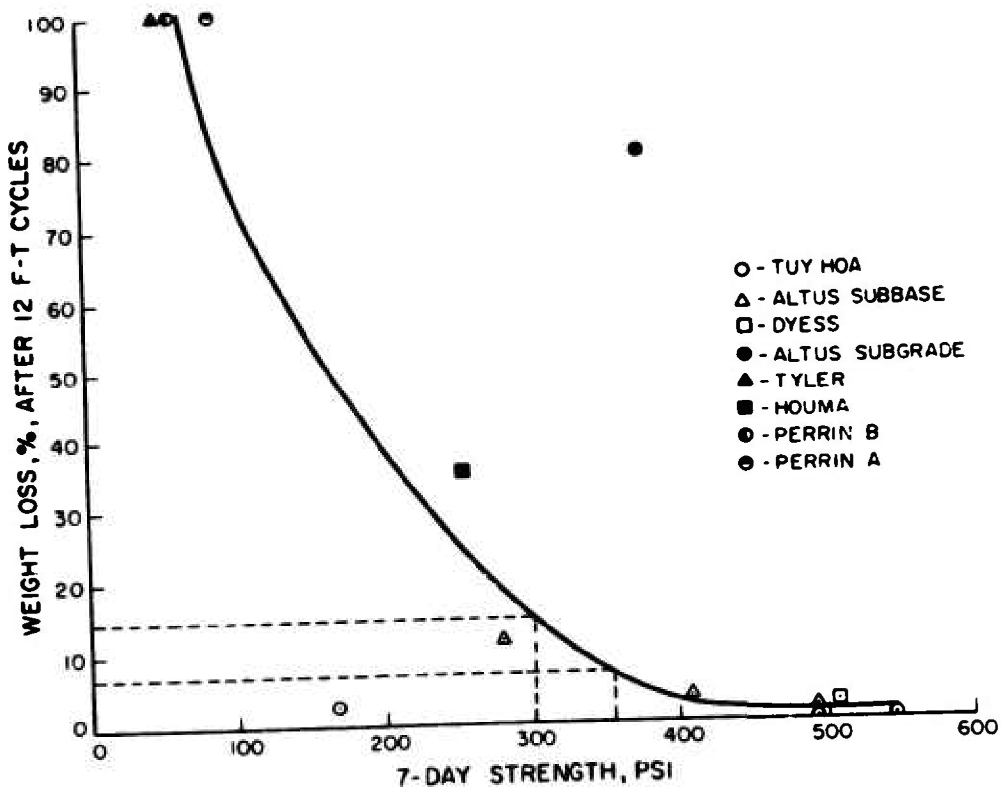


Figure 100 - Relation Between 7-Day Strength and Weight Loss After 12 Freeze-Thaw Cycles for Soil-Cement Mixtures

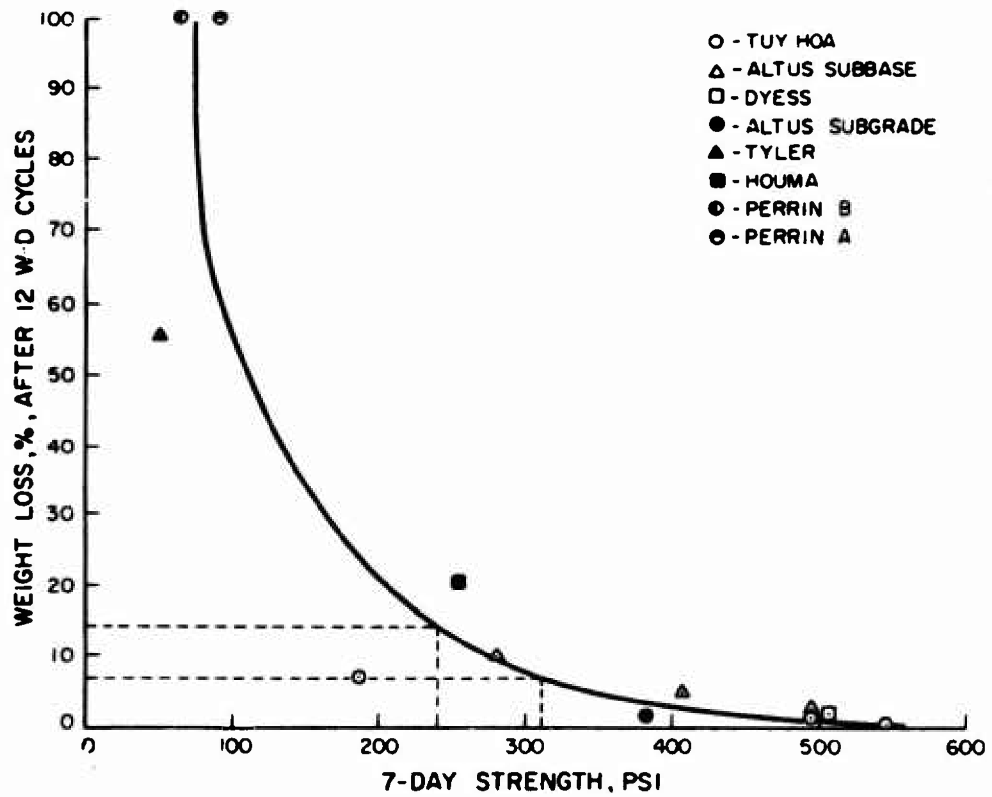


Figure 101 - Relation Between 7-Day Strength and Weight Loss After 12 Wet-Dry Cycles for Soil-Cement Mixtures

A nonlinear relationship was obtained between the residual strength after 12 freeze-thaw cycles and 7-day strength (Figure 102). This relationship indicates that the rate of increase in residual strength was faster if the 7-day strength was approximately 400 psi or more, and there would be no residual strength if the 7-day strength was about 60 psi. The relationship between residual strength after 12 wet-dry cycles and 7-day strength was linear (Figure 103) as shown by the equation:

$$q_w = -149.33 + 3.107\ q_7$$

A soil-cement specimen having a 7-day strength of about 50 psi or less appeared to have no residual strength after 12 wet-dry cycles.

A linear relationship was also found between long-term immersion durability (as measured by long-term immersed strength) and 7-day strength (Figure 104). The linear equation is as follows:

$$q_{ii} = -32.63 + 1.528\ q_7$$

The various relationships obtained by use of this limited data show that the use of 7-day strength is very promising to predict 28-day strength and durability of soil-cement mixtures.

11. Relationship Between Durability Tests

The selection of the type of durability test to be performed on soil-cement specimens will often be governed by the type of equipment available. From this standpoint, it is of interest to know whether there is any relationship between the various durability tests. The relationships found for the soils used in this research are discussed below.

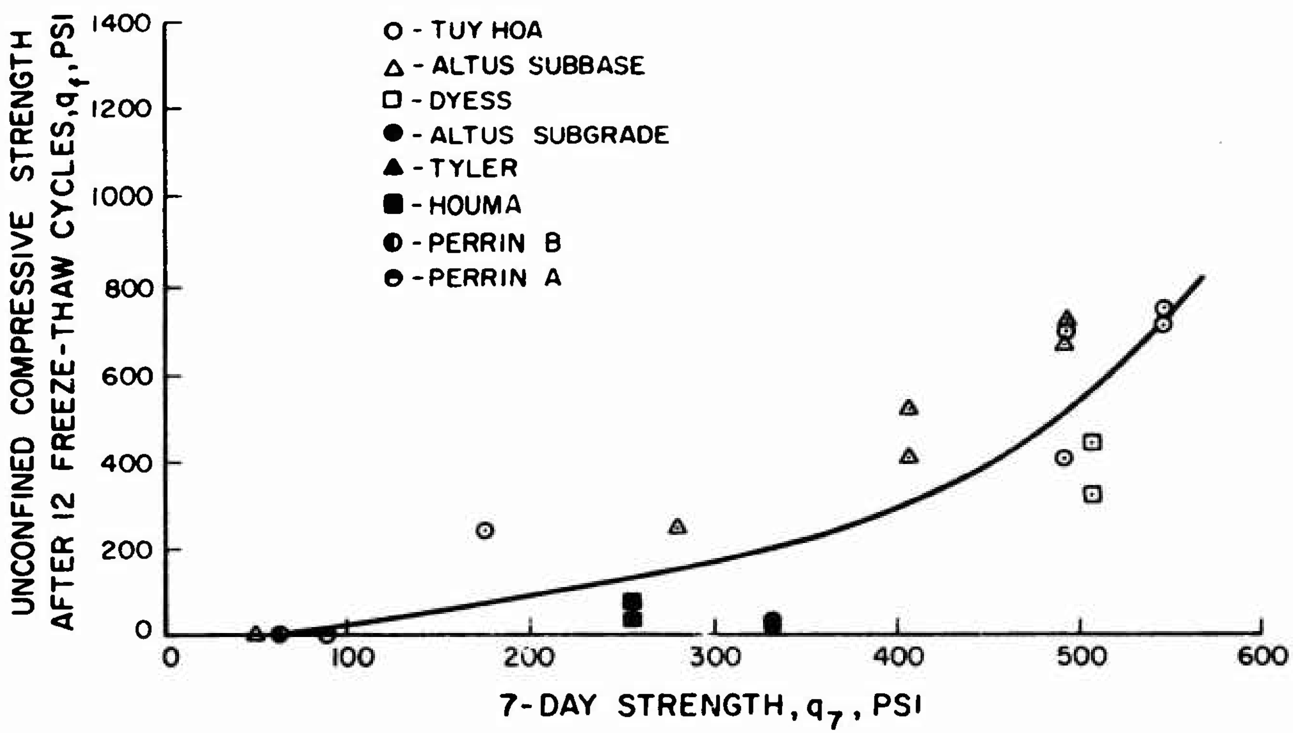


Figure 102 - Relation Between 7-Day Strength and Residual Strength After 12 Freeze-Thaw Cycles for Soil-Cement Mixtures

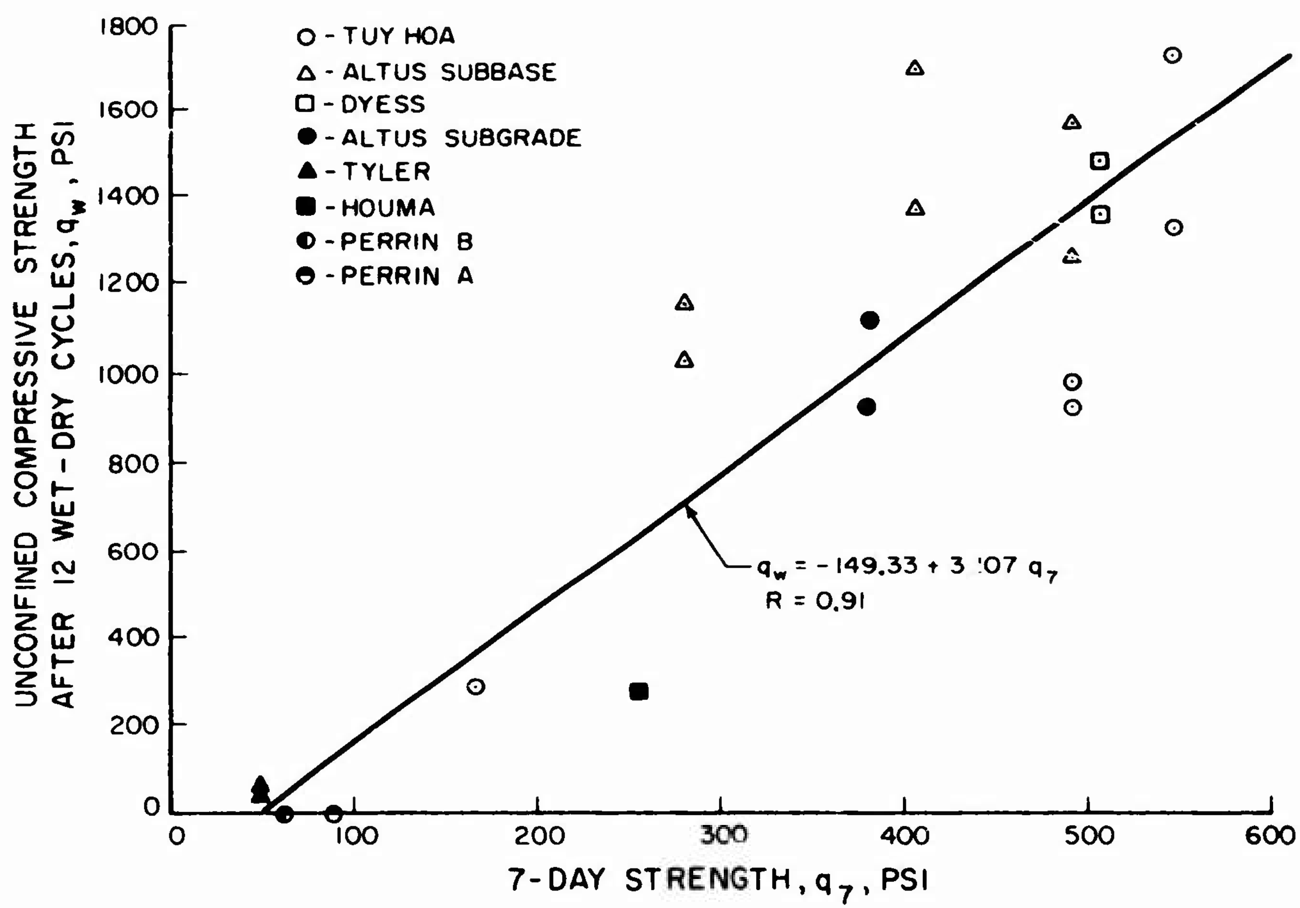


Figure 103 - Relation Between 7-Day Strength and Residual Strength After 12 Wet-Dry Cycles for Soil-Cement Mixtures

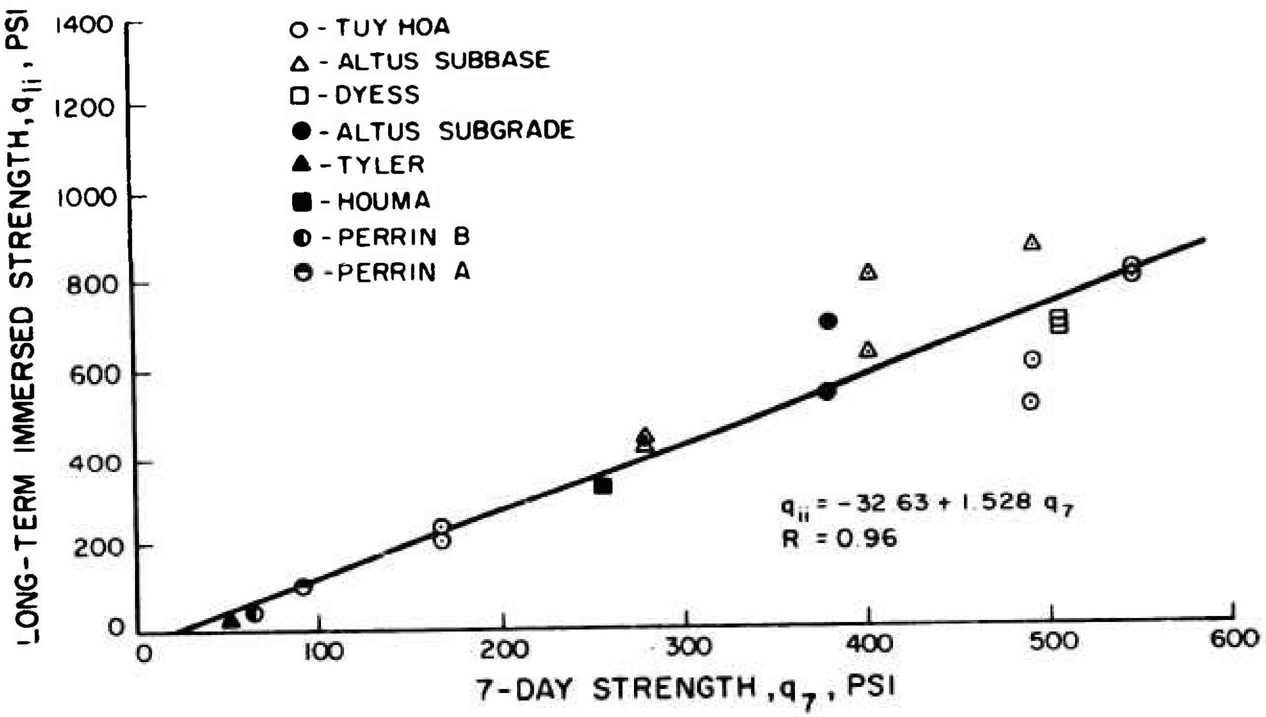


Figure 104 - Relation Between 7-Day Strength and Long-Term Immersed Strength for Soil-Cement Mixtures

a. Weight loss versus residual strength after freeze-thaw and wet-dry cycles.

The relation between weight loss and residual strength after 12 freeze-thaw or 12 wet-dry cycles is shown in Figures 105 and 106. The test data used to establish these curves are limited and some scatter of results was observed. On the basis of the PCA specifications for weight loss, Figures 105 and 106 indicate that the following residual strength-weight loss relationship exists:

Freeze-Thaw Test

Allowable weight loss	Residual strength required
14 percent	200 psi
7 percent	280 psi

Wet-Dry Test

Allowable weight loss	Residual strength required
14 percent	280 psi
7 percent	640 psi

b. Immersed strength versus freeze-thaw and wet-dry durability.

The relationship between the immersed strength and freeze-thaw durability is presented in Figure 107, whereas Figure 108 shows the relationship between immersed strength and wet-dry durability. Durability was expressed as the unconfined compressive strength after 12 wet-dry or freeze-thaw cycles.

The relationship between long-term immersed strength and durability after 12 freeze-thaw cycles was nonlinear (Figure 107), but a linear relationship was obtained between long-term immersed strength and wet-dry durability (Figure 108). Although more data are needed for conclusive results, it appears that long-term immersed strengths may be a promising method of predicting freeze-thaw and wet-dry durability.

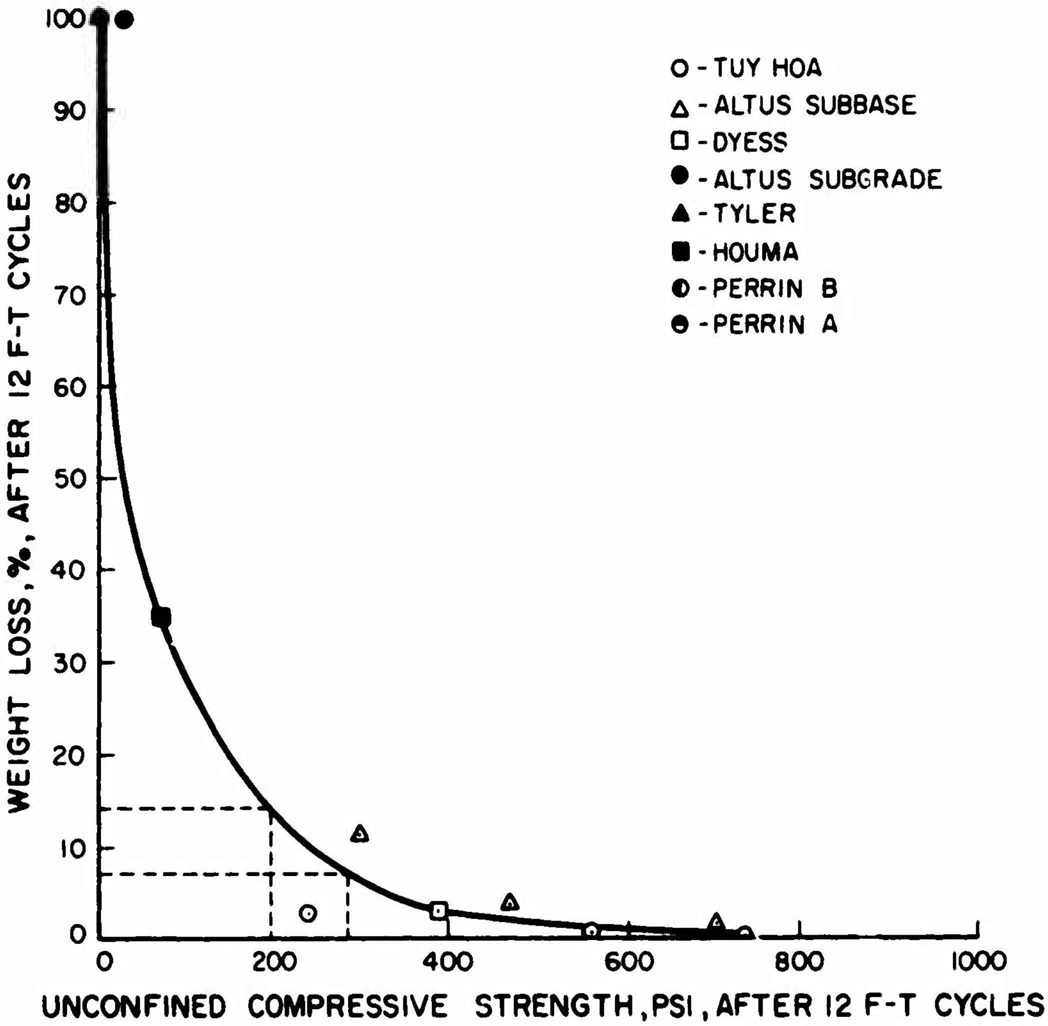


Figure 105 - Relation Between Residual Strength and Weight Loss After 12 Freeze-Thaw Cycles for Soil-Cement Mixtures

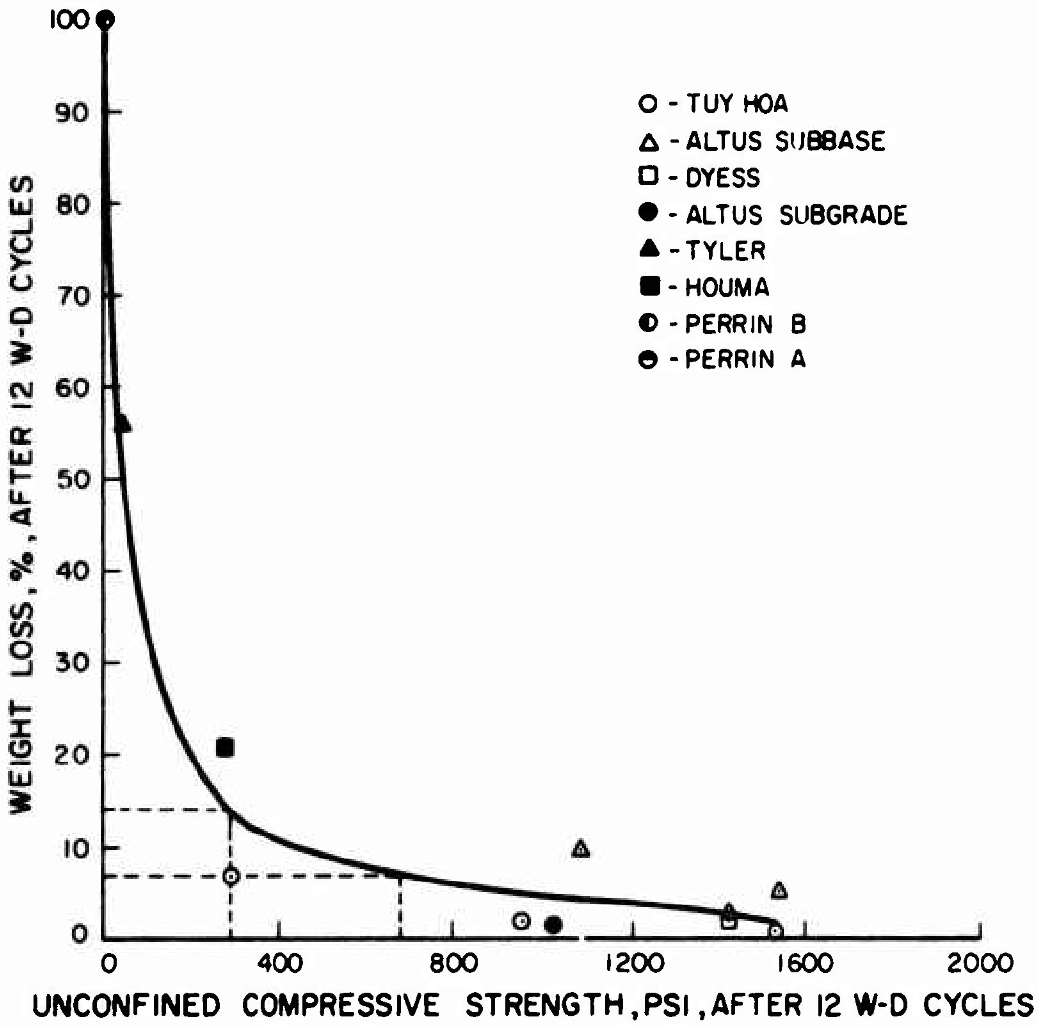


Figure 106 - Relation Between Residual Strength and Weight Loss After 12 Wet-Dry Cycles for Soil-Cement Mixtures

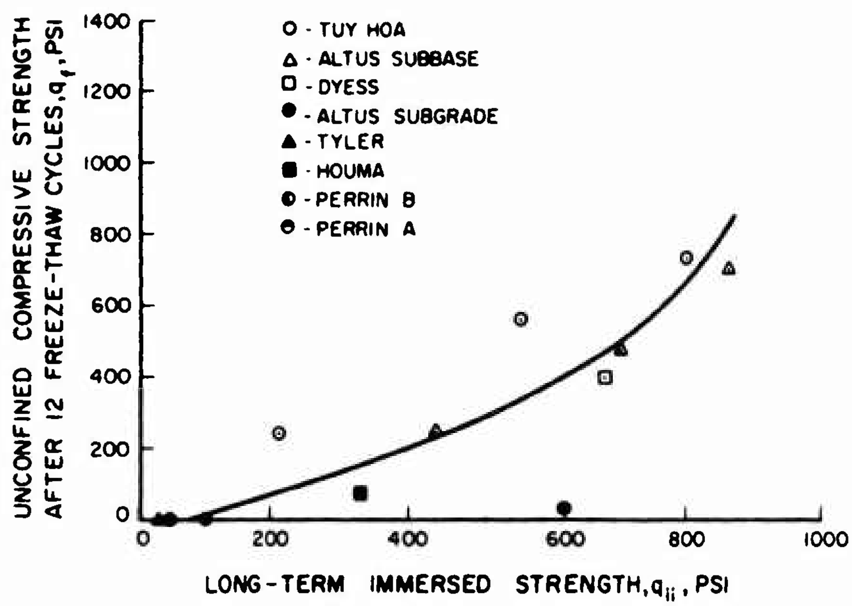


Figure 107 - Relation Between Long-Term Immersed Strength and Residual Strength After 12 Freeze-Thaw Cycles for Soil-Cement Mixtures

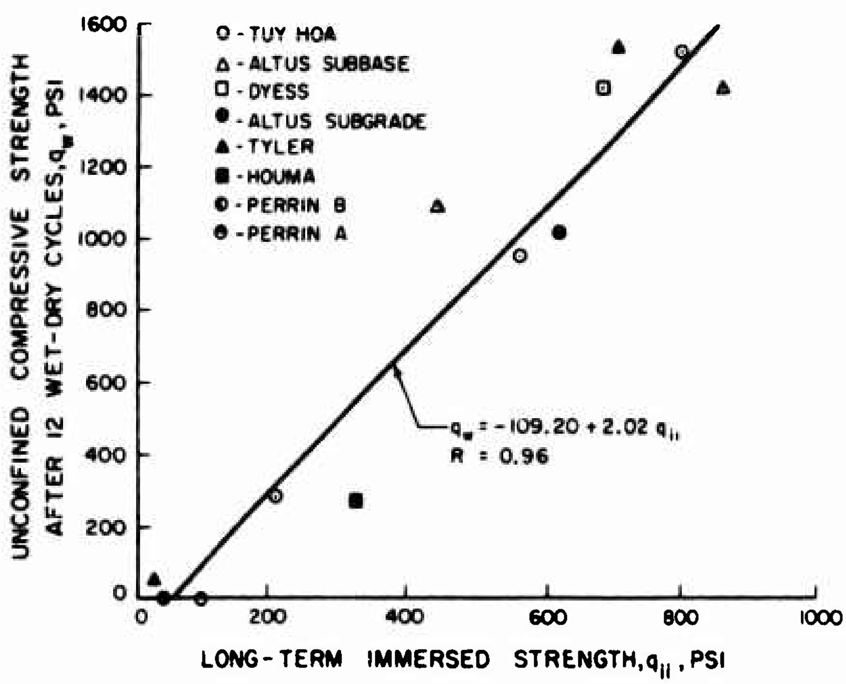


Figure 108 - Relation Between Long-Term Immersed Strength and Residual Strength After 12 Wet-Dry Cycles for Soil-Cement Mixtures

12. Validity of pH Test for Determining Harmful Organic Content

This investigation was made to check the validity of the pH test proposed by Maclean and Sherwood (reference 22) for detecting harmful organic matter interfering with cement stabilization. Figure 109 shows the relationship between the pH of the soil-cement mixtures (10:1) and the 7-day strength of soil-cement specimens molded at their median cement contents.

The highly organic soils, Tuy Hoa, Tyler and Houma, gave pH values below 12.1 after addition of 10 percent cement and water. Of these three soils, Tuy Hoa exhibited an unusually high 7-day strength when treated with cement. The other two were not successfully stabilized, and it appeared that the organic matter was active enough to interfere with cement stabilization.

The data for Tuy Hoa, when plotted with the original data of Maclean and Sherwood (not shown herein), fall far out of the range of soils they encountered. Hence, it is considered to be an unusual soil. Tyler was an acidic soil (pH = 2.3), and it is not known whether acidity resulting from oxidation of pyrite (FeS) in the soil contributed to its deterioration in strength. However, a previous investigation (reference 22) showed that for acid soils, both high and low soil-cement strengths were obtained.

Figure 109 further shows that Altus subbase, Dyess and Altus subgrade mixtures, with pH of 12.1 or more, exhibited high 7-day strengths. On the other hand, the sulfate bearing soils, Perrin B and Perrin A, had a pH of 12.1 or more and were not stabilized.

From the above test data it can be concluded that if the pH test value was below 12.1 in a soil-cement mixture, it indicated the presence of active organic matter in the soil interfering with hardening of soil-cement (with the exception of an unusual soil, Tuy Hoa). It might also indicate the presence

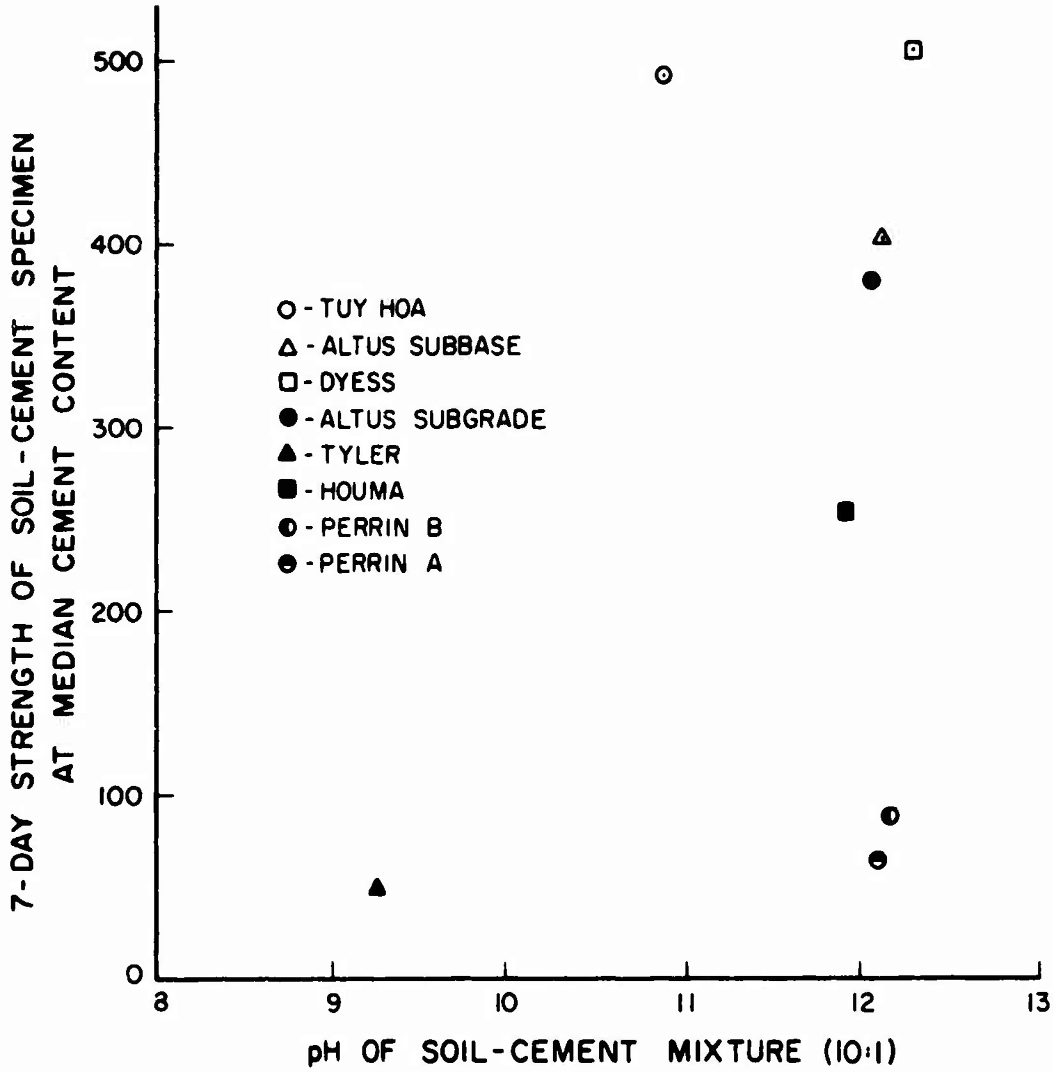


Figure 109 - Relation Between pH and 7-Day Strength for Soil-Cement Mixtures

of acid-producing inorganic compounds that could affect normal hardening of cement-treated soils. Thus the pH test criteria can be modified slightly as follows: if the test gives a pH below 12.1 in a soil-cement mixture, it indicates the presence of harmful organic matter in the soil interfering with cement stabilization of soil and/or acid-producing inorganic compounds that might also affect normal hardening of soil-cement. However, a pH value higher than 12.1 does not necessarily ensure satisfactory hardening of cement-stabilized soils.

13. Changes in Existing Subsystems

Based on the research discussed above, some minor changes were made in the cement stabilization subsystem. The change which appeared most needed was to get the pH test for harmful organics into the expedient subgrade and base course subsystems. Since the lime pH test has already been recommended, it seems reasonable to assume that a pH meter will be available for the cement determinations also. The use of the pH test to quickly eliminate soils which will not stabilize will do much to hasten an already slow process.

The time required to evaluate soils for cement stabilization is one of the major drawbacks in the expedient subsystems. An attempt was made to develop an accelerated curing procedure for cement stabilized soils similar to that reported for lime stabilized soils. There was a complete lack of success in the procedures used and the results are not reported herein. It was found that high temperatures did accelerate the cure, but the accelerated curing times required to reach 7 or 28 day strengths varied considerably according to the soil type, and standard accelerated curing times could not be obtained. This is certainly a fruitful area for future research.

Until such time as a suitable accelerated curing procedure is developed,

little can be done to improve on the Portland Cement Association evaluation procedures except to change some of the compressive strength limits to make them coincide with those obtained in this research. This has been done and the subsystems in Appendixes A through D reflect the changes made.

SECTION V

BITUMINOUS STABILIZATION SUBSYSTEM

1. Introduction

Originally, SSIS indicated that for a soil to be suitable for bituminous stabilization the plasticity index should be below 10, and it should contain less than 25 percent by weight passing the number 200 sieve. These requirements, based on an extensive literature review (reference 4), were predicted on the mixing capability of existing equipment, mixture strength, and the amount and therefore expense of bituminous material required to bind and waterproof the soil.

For validation purposes, a laboratory testing program to determine the suitability of these requirements was performed. This testing program consisted of laboratory mixing to determine which soils could be suitably combined with bitumen followed by strength and durability evaluations of selected soils. The testing program is described below.

2. Bituminous Mixing Tests

Soils whose physical properties were described in Section II were mixed with a slow setting anionic emulsion. Handmixing with a spoon in a bowl was utilized as this is a reasonable method of simulating the combination of the soil and bitumen in road mixing operations. Results of this series of mixing tests are presented in Table 14.

The ability of the soil to be mixed with the emulsion was rated according to the following categories: very good, good, fair, poor and very poor. These ratings generally define the ease of mixing and distribution of the bitumen.

TABLE 14

BITUMINOUS MIXING TESTS

Soil Identification	Ability to be Mixed	
	Rating	Description
Altus Subgrade-Not processed	Poor	Asphalt poorly distributed
Altus Subbase-Not Processed	Fair	Asphalt well distributed, somewhat difficult to mix
East Texas	Good	Asphalt well distributed, fairly easily mixed
Brazos	Very Good	Asphalt well distributed, easily mixed
Bryan	Very Good	Asphalt well distributed, easily mixed
Chenault	Very Poor	Asphalt poorly distributed
Dallas Regional Airport	Poor	Asphalt poorly distributed
Dallas Western	Poor	Asphalt poorly distributed
Dansby	Fair	Asphalt distributed but somewhat difficult to mix
Dyess	Fair	Asphalt not distributed
North Carolina-Processed	Poor	Asphalt poorly distributed
North Carolina-Not Processed	Poor	Asphalt poorly distributed
Panama (A)-Processed	Poor	Asphalt poorly distributed
Panama (A)-Not Processed	Very Poor	Asphalt poorly distributed
Panama (B)-Not Processed	Poor	Asphalt poorly distributed
Perrin A-Processed	Very Poor	Asphalt poorly distributed
Perrin B-Processed	Very Poor	Asphalt poorly distributed
Tuy Hoa	Very Good	Asphalt well distributed, easily mixed
Tyler	Poor	Asphalt poorly distributed
Yoakum	Very Good	Asphalt well distributed, easily mixed
WES-Buckshot Clay	Very Poor	Asphalt poorly distributed
WES-Lean Clay	Poor	Asphalt poorly distributed
WES-Gravel	Very Good	Asphalt well distributed, easily mixed
WES-Sandy Gravel	Good	Asphalt well distributed, fairly easily mixed

A general description accompanies this rating on Table 14. Various percentages of water and emulsions were added to the soils until the optimum mixing conditions as determined by visual examination and ease of hand mixing could be achieved. Emulsion contents were limited, however, to 10 percent by dry weight of soil. Water contents ranged from 0 to 30 percent by dry weight of soil.

Based on the mixing tests presented above and information available in the literature, as summarized in reference 4, the present criteria for bituminous stabilization will ensure that roadway mixing operations will be possible with existing equipment. Mixing in central plants, either continuous or batch type, is normally better than road mixing; thus, the existing criteria suggested in SSIS should ensure adequate bitumen-soil mixing in all cases provided good construction procedures are utilized.

3. Strength and Durability Tests

Those soils which were considered to be capable of being mixed with bitumen were subjected to strength and durability tests. Asphalt cements, cutback asphalts, and emulsions were utilized in this testing program. The properties of the bitumens used are described in reference 25.

Results of Marshall tests of 9 materials are shown in Table 15. Durability tests were performed according to a proposed ASTM test method (reference 43) which utilizes samples soaked for 24 hours in a 140°F water bath. The Marshall test is conducted after this soaking period and the results are compared with Marshall strength values determined on companion samples tested in the standard manner. Results are shown in Table 16.

a. Mixtures containing asphalt cements.

According to Air Force requirements (reference 28) (Table 17) for asphalt concrete and sand asphalt mixtures, Marshall stability and flow values for the

TABLE 15

RESULTS OF MARSHALL TESTS ON BITUMINOUS STABILIZED SOILS

Soil Identification	Asphalt Type	Asphalt Content, percent*	Marshall Stability, lbs.**	Marshall Flow, 0.01 in.	Air Void Content, percent
Altus Subbase	Asphalt Cement	3.0	1700	11	22.3
East Texas	Asphalt Cement	7.5	950	9	8.5
East Texas	Cutback	7.1	1020	16	---
East Texas	Anionic Emulsion	8.7	140	19	---
Brazos	Cationic Emulsion	6.3	550	9	---
Bryan	Asphalt Cement	3.8	670	11	2.4
Bryan	Cutback	3.9	50	20	---
Bryan	Anionic Emulsion	4.6	950	10	---
Dansby	Asphalt Cement	4.7	150	14	11.1
Dansby	Cutback	7.2	130	13	---
Dansby	Anionic Emulsion	9.4	450	13	---
Tuy Hoa	Asphalt Cement	1.5	220	14	28.0
WES - Gravel	Asphalt Cement	3.1	780	15	16.0
WES - Sandy Gravel	Cationic Emulsion	9.4	940	9	---
Yoakum	Cationic Emulsion	9.4	620	10	---

*Percent by dry weight of aggregate.

**Strengths were obtained after mixing and compacting, tested at 140°F.

TABLE 16

DURABILITY TEST RESULTS

Aggregate	Asphalt Type	Resistance of Bituminous Mixture to the Effect of Water, R_R*
East Texas	Asphalt Cement	79
WES-Gravel	Asphalt Cement	100
Dansby	Asphalt Cement	85
Bryan	Asphalt Cement	98
East Texas	Cutback Asphalt	71

$$*R_R = \frac{S_2}{S_1} \times 100$$

S_1 = average of Marshall stability values for mixtures not subjected to water

S_2 = average of Marshall stability values for mixtures subjected to 140°F water bath for 24 hours

TABLE 17

CRITERIA FOR DETERMINATION OF OPTIMUM BITUMEN CONTENT
(Marshall Method)

Test Property	Type of Mix	Point on Curve: For 100 psi tires[1]	Point on Curve: For 200 psi tires[1]	Criteria: For 100 psi tires[1]	Criteria: For 200 psi tires[1]
Stability	Asphaltic-concrete surface course	Peak of curve	Peak of curve	500 lb or higher	1800 lb or higher
	Asphaltic-concrete binder course	Peak of curve[2]	Peak of curve[2]	500 lb or higher	1800 lb or higher
	Sand asphalt	Peak of curve		500 lb or higher	
Unit weight	Asphaltic-concrete surface course	Peak of curve	Peak of curve	Not used	Not used
	Asphaltic-concrete binder course	Not used	Not used	Not used	Not used
	Sand asphalt	Peak of curve		Not used	Not used
Flow	Asphaltic-concrete surface course	Not used	Not used	20 or less	16 or less
	Asphaltic-concrete binder course	Not used	Not used	20 or less	16 or less
	Sand asphalt	Not used	Not used	20 or less	16 or less
Percent voids total mix	Asphaltic-concrete surface course	4 (3)	4 (3)	3-5 (2-4)	3-5 (2-4)
	Asphaltic-concrete binder course	5 (4)	6 (5)	4-6 (3-5)	5-7 (4-6)
	Sand asphalt	[illegible] (5)	-- (-)	5-7 (4-6)	--- (---)
Percent voids filled with bitumen	Asphaltic-concrete surface course	80 (85)	75 (80)	75-85 (80-90)	70-80 (75-85)
	Asphaltic-concrete binder course	70 (75)	60 (65)[2]	65-75 (70-80)	70-80 (55-75)
	Sand asphalt	70 (75)	-- (--)	65-75 (70-80)	-- (--)

[1] Figures in parentheses are for use with bulk impregnated specific gravity (water absorption greater than 2.5 percent).

[2] If the inclusion of asphalt contents of these points in the average causes the voids to fall outside the limits, then the optimum asphalt content should be adjusted so that the voids total mix are within the limits.

(from reference 26)

Altus subbase, East Texas, Bryan and both WES gravel materials tested in this program are within limits specified for 100 psi tires. The materials which can be classified as sands (Dansby and Tuy Hoa), however, did not possess 500 pound minimum stability recommended in all cases. Additionally, the air void content was in excess of the range specified.

The criteria for Marshall properties as utilized by the Air Force are based on surface course requirements for mixtures containing asphalt cements. These requirements may be in excess of those required for base and subgrade stabilization as the temperature and the stress in base and subgrade are normally below those of the surface course. Thus, the testing temperature and/or the Marshall stability and flow requirements should be adjusted for realistic base and subgrade temperature and load conditions.

As a first approximation and until additional laboratory and in-service data are collected, the following method of bituminous stabilized base course mixture design is recommended.

i. Selection of Asphalt Quantity

Selection of asphalt contents for bituminous mixtures should be determined utilizing current Corps of Engineers methods. Thus, a testing temperature of 140°F should be used to establish stability and curves upon which the design asphalt content is, in part, based.

ii. Suitability of Mixtures

Marshall stability and flow characteristics at the design asphalt content as selected above should be determined at a temperature representative of the base course's maximum temperature measured at one-third of the distance from the top of base course. (The maximum temperature should occur less than about 5 percent of the time during any given year.) Field measurements of temperature from nearby pavements is the preferred method

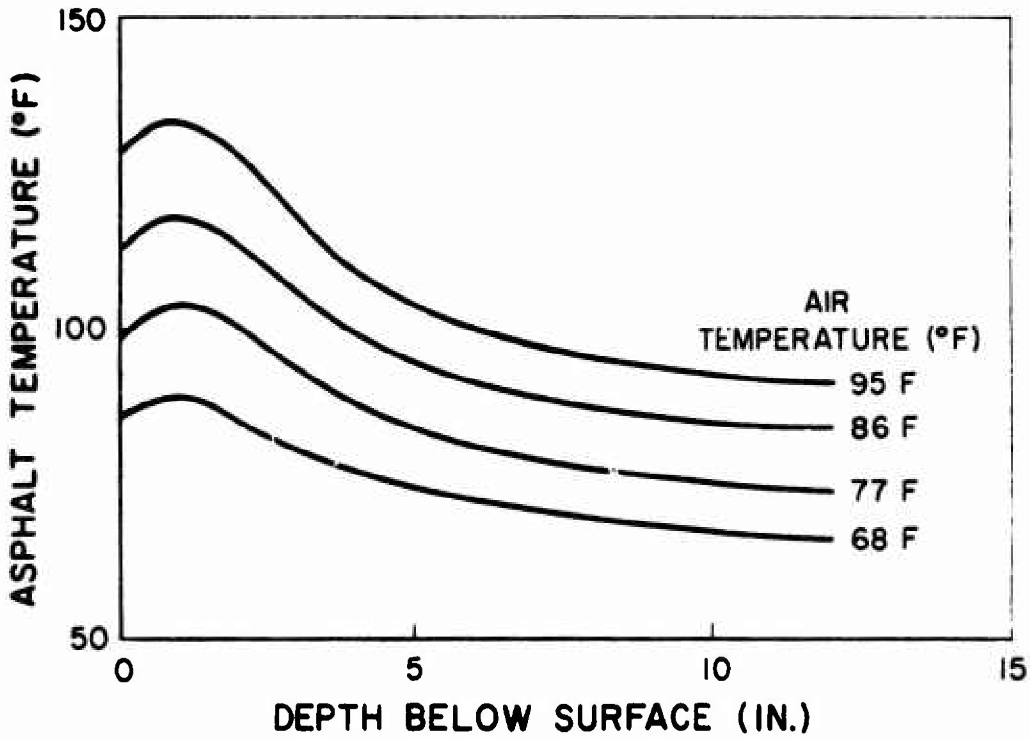
for obtaining the necessary temperature data. If this is not possible, the next recourse is to calculate a value by the methods given in references 27 and 28. If either of these two methods can not be used, then determine a value from Figure 110. Finally, under the most expedient situations, use 100°F for the cool, northern climates or 120°F for the hotter, southern climates of the Northern Hemisphere.

Consideration should be given to altering the requirements for percent voids and percent voids filled with bitumen. Current state requirements (reference 29) and Asphalt Institute recommendations (reference 30) could be utilized as a basis for development of these revised criteria. Criteria suggested for present usage are shown in Table 18.

b. Mixtures containing liquid asphalts.

Test methods and acceptance criteria for soil mixtures containing liquid asphalts (cutbacks and emulsions) are not well established. A variety of methods which include Hubbard-Field test, Hveem stability, Marshall stability, Florida bearing test, Iowa bearing test, extrusion test, unconfined compression test, triaxial compression test, "R" value, and elastic modulus have been utilized. The most promising tests for utilization by the Air Force include the Marshall, Hveem, and extrusion tests. Criteria for the Marshall and Hveem tests have been developed by several investigators and are reviewed in reference 4.

For this research, a proposed Marshall testing procedure as published by ASTM (reference 31) was utilized for mixtures containing cutbacks and emulsions. Results are given in Table 15. As indicated in reference 25, the amount of curing prior to testing is critical if adequate mixture strengths are to be obtained. The proposed ASTM test method suggests that testing should be performed on cutbacks after placing the specimens in a 140°F oven for 2 hours.



(from reference 37)

Figure 110 - Relation of Temperature of Asphalt Surface to Depth Below Surface

TABLE 18

SUGGESTED MARSHALL MIX DESIGN CRITERIA*

TEST PROPERTY	TIRE PRESSURE	
	100 PSI	200 PSI
Stability, pounds	500 or more	1800 or more
Flow, 0.01 inch	20 or less	16 or less
Air Voids	4-6	5-7
Percent Voids Filled with Asphalt	65-75	70-80

*Tests for compliance to criteria should be conducted at a temperature representative of a critical in-service condition.

Compaction of the specimen is to take place after 50 percent of the volatiles have escaped. These compaction, curing and testing conditions may not be optimal for ultimate Air Force use but appear to be the best methods currently available. The same Marshall test method was utilized for testing of emulsion mixtures although the method was not specifically developed for emulsions. This method is suggested for use by the Air Force on an interim basis.

Design criteria for mixtures containing liquid asphalts should be identical to those for asphalt cement mixtures until more realistic criteria can be developed. Curing conditions as stated above are critical when evaluating the mixture properties. Since field compaction usually proceeds when about 50 percent of the volatiles from the liquid asphalt has evaporated from the mixture, the proposed ASTM test method including its curing procedure appears appropriate.

Results from the cutback and emulsion mixture testing program also indicated that in SSIS the equation for determining the amount of cutback (page 58, reference 4) and the table utilized to determine the amount of emulsion (page 61, reference 4) are not sufficiently accurate for mixture design purposes. However, they do serve as reliable starting points for the selection of asphalt quantities to be verified by laboratory and/or field trial mixtures. If a laboratory testing program will not be utilized to establish bitumen content for the mixture, experienced personnel should be utilized at the construction site to make the necessary adjustments in bitumen content.

4. Bituminous Stabilization Subsystem Revisions

Based on the series of tests discussed above together with information summarized since the initial preparation of SSIS, revised flow diagrams and tables have been suggested for the bituminous stabilization subsystems. The

flow diagrams are shown below in Figures 111 through 114. The complete flow diagrams and accompanying tables are presented in Appendixes A through D.

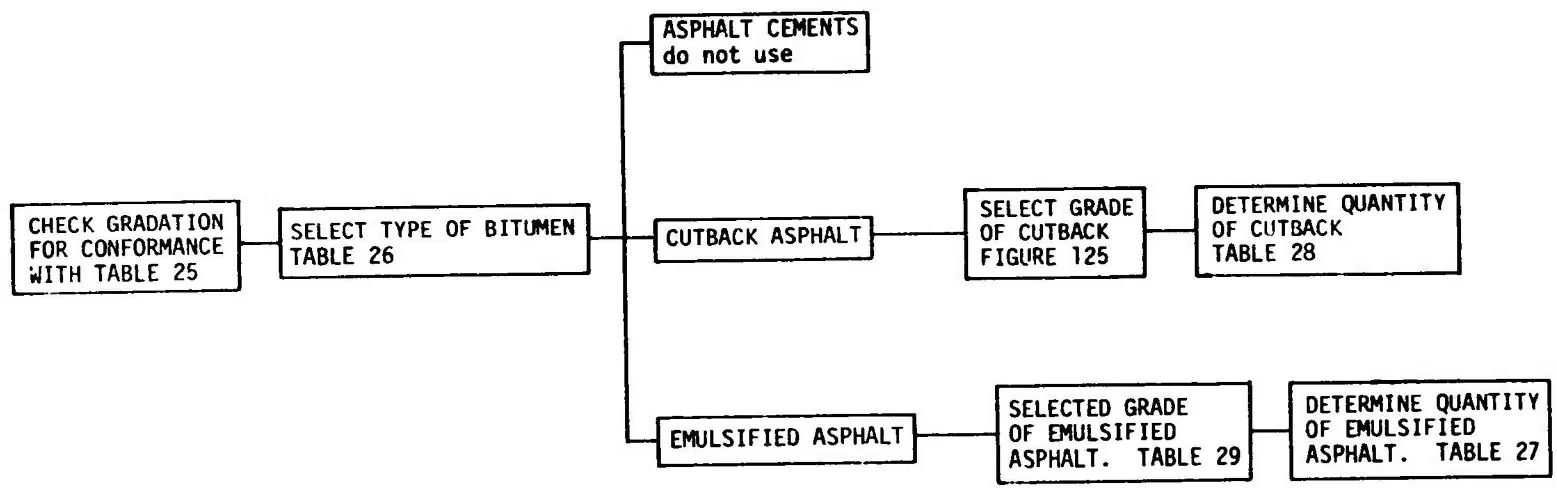


Figure 111 - Subsystem for Expedient Subgrade Stabilization with Bituminous Materials

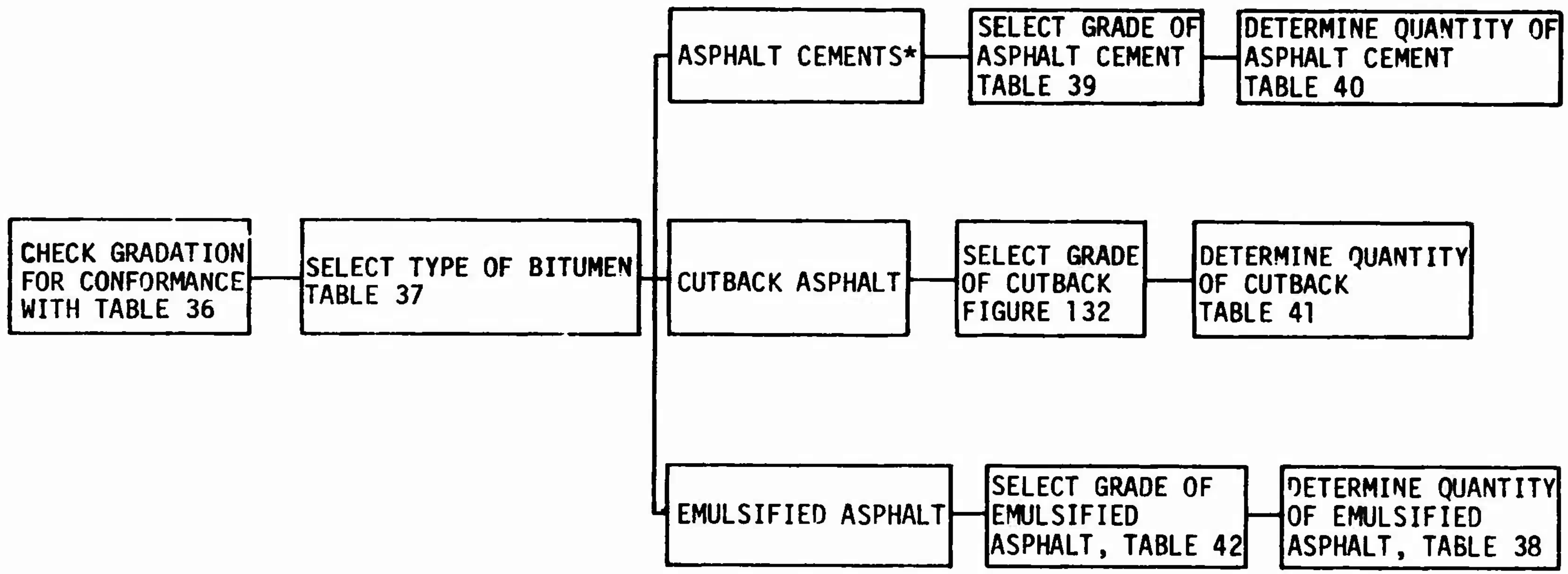


Figure 112 - Subsystem for Expedient Base Course Stabilization with Bituminous Materials

*Hard asphalt cements are preferred in hot climates.

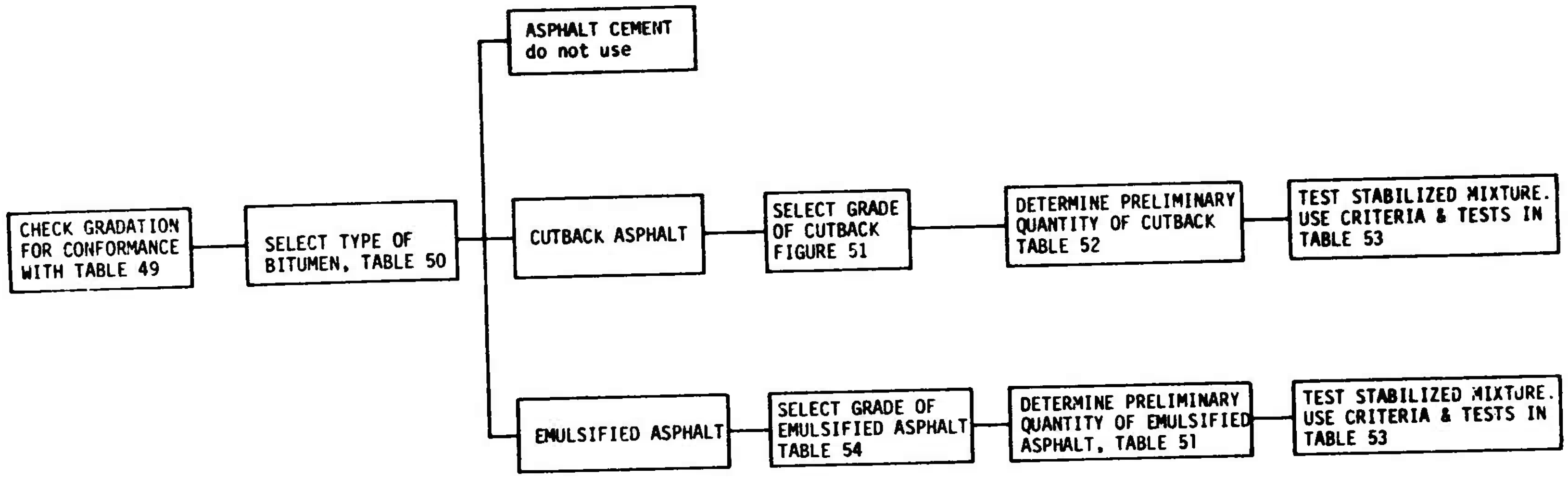


Figure 113 - Subsystem for Nonexpedient Subgrade Stabilization with Bituminous Materials

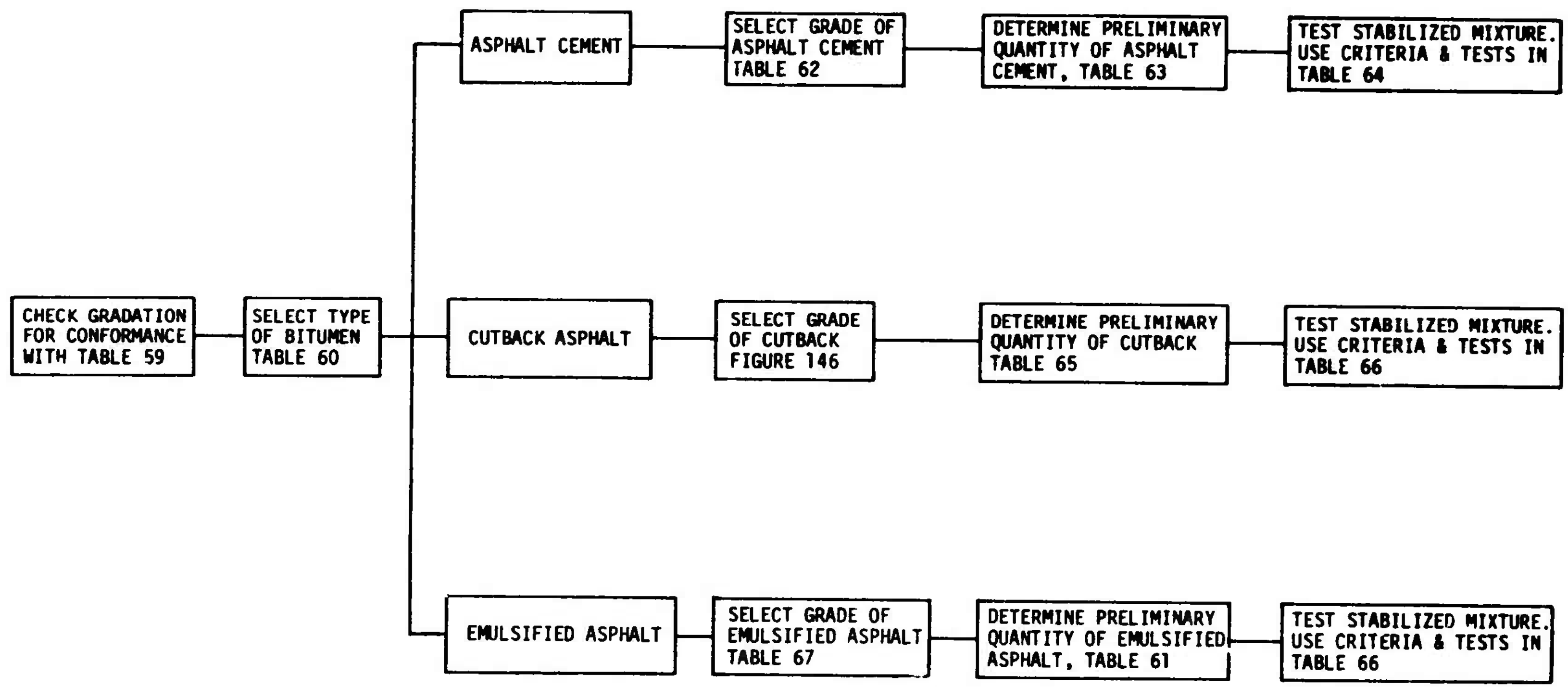


Figure 114 - Subsystem for Nonexpedient Base Course Stabilization with Bituminous Materials

SECTION VI

DYNAMIC TESTING OF STABILIZED MATERIALS

1. Introduction

Many current pavement design methods do not adequately consider the structural benefits gained from stabilized layers. The newer rational approach to pavement design such as those utilizing layered elastic theory have the capability of ascertaining the increased load carrying ability of stabilized materials. However, certain material constants and failure criteria need to be developed. Realizing this need for research, several agencies have attempted to add to the body of knowledge necessary for the development of the newer pavement design techniques.

A review of the literature associated with rational pavement design and materials characterization together with detailed information on materials and dynamic tests performed on three stabilized materials can be found in reference 32. Test methods and test results from reference 32 are summarized below. This information is presented with the hope that it will form the genesis for combining a pavement design system with the soil stabilization index system.

2. Test Methods

A number of test methods have been utilized to determine elastic constants and failure criteria for stabilized materials. Triaxial compression, rotating cantilevers, beams, cylinders and plates are some of the more common methods of tests. These test methods allow the determination of elastic constants as well as fatigue failure criteria. Third point loading on beam specimens and single point loading on circular plates were utilized in this study.

In addition to the type of test, the loading pattern was also investigated as previous research had indicated the load pattern will affect the test results. Full-wave and half-wave sine loading were utilized in the testing program as described below:

a) beam specimen, full-wave sine loading

b) beam specimen, half-wave sine loading

c) circular plate specimen, full-wave sine loading

3. Asphalt Stabilization Test Results

The asphalt concrete mixture tested consisted of an asphalt cement (AC-20) mixed with a poorly graded crushed sandstone. This mixture was utilized in a pavement at a test site near Big Brown, Texas. The pavement carries 200 ton capacity lignite hauling vehicles. Test sections were constructed to furnish information on the performance of stabilized materials under heavy wheel loads. Detailed analysis of these test sections can be found in references 32 and 33.

Fatigue test results of this mixture under various forms of loading are shown in Figures 115 and 116 for the stress-fatigue life and strain-fatigue life relationships. A comparison with published data (reference 33) is included on the figures.

Resilient modulus of elasticity, density and air void content of mixtures tested are given in Table 19. These values were obtained at a test temperature of 70°F and a loading rate of 100 cycles per minute. The magnitude of these test results agree with values published in the literature.

Based upon the test results of the asphalt stabilization testing program and the extensive results presented in reference 32 the following general conclusions appear warranted:

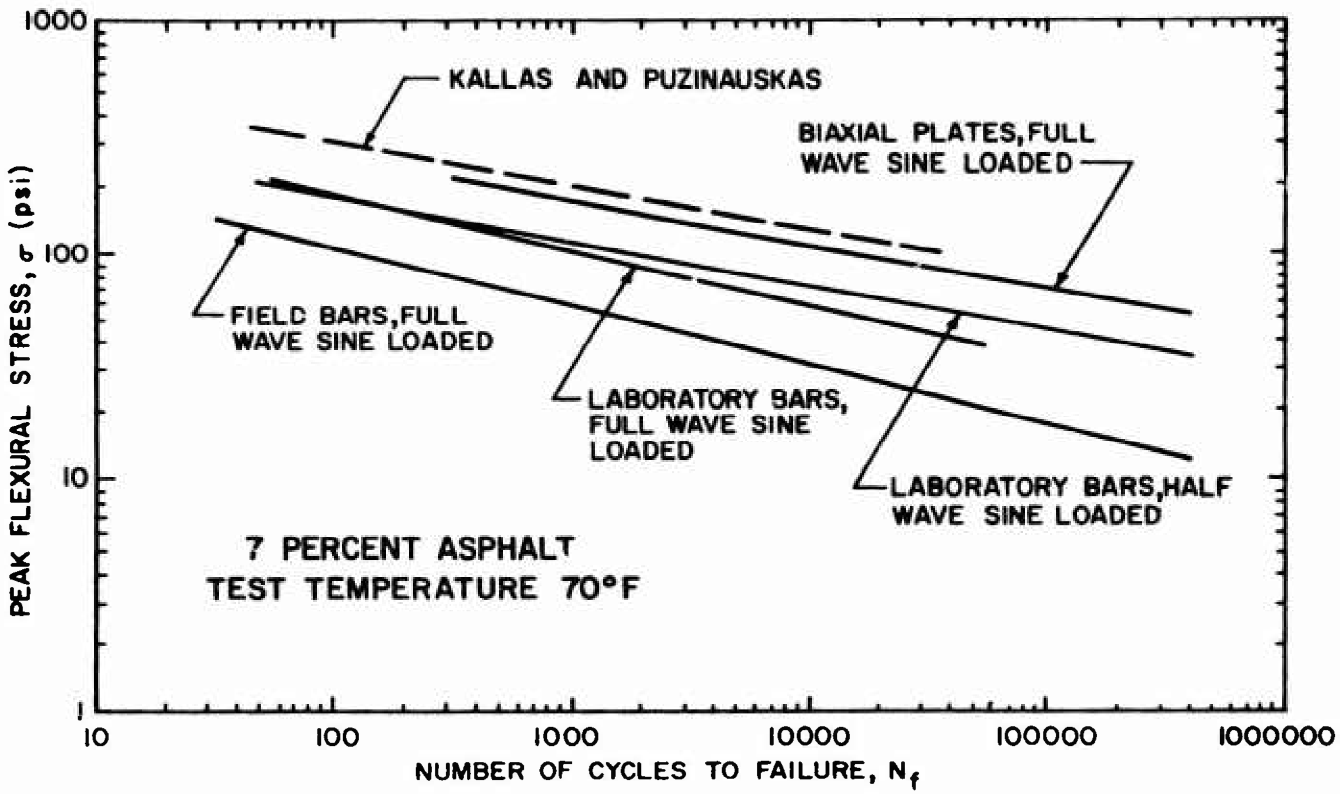


Figure 115 - Stress Fatigure Curves for Asphalt-Stabilized East Texas Crushed Stone

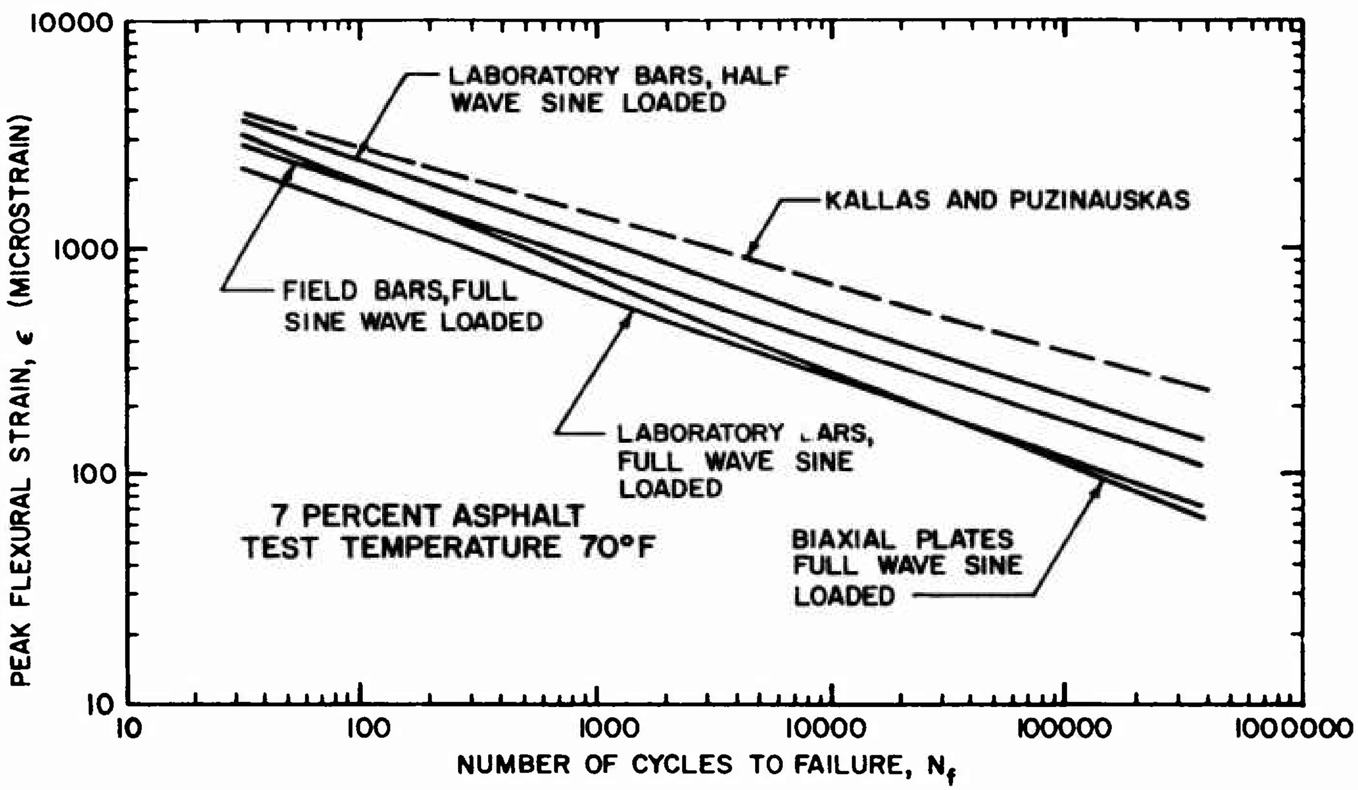


Figure 116 - Strain Fatigure Curves for Asphalt-Stabilized East Texas Crushed Stone

TABLE 19

AVERAGE RESILIENT MODULUS OF ELASTICITY, AVERAGE DENSITY,

AND AVERAGE AIR VOIDS FOR ASPHALT-STABILIZED MATERIALS

Material and Test Method	Average Resilient Modulus of Elasticity, psi	Average Density lb./cu.ft.	Average Air Voids, percent
Laboratory Beams Full-Wave Sine Loaded	233,000	126.1	15.0
Laboratory Beams Half-Wave Sine Loaded	166,000	125.8	15.2
Field Beams Full-Wave Sine Loaded	84,000	120.5	18.8
Laboratory Plates Full-Wave Sine Loaded	261,000	125.3	15.5

a) Fatigue results obtained on laboratory prepared specimens closely correspond to results obtained on specimens taken from the field section.

b) A statistical comparison of the effect of test method on fatigue life of this asphalt stabilized mixture indicated the results from half-wave sine and full-wave sine loading significantly differ.

c) Statistical comparison of regression parameters from uniaxial and biaxial full-wave sine loading fatigue tests was mixed but suggested that a significant difference exists.

d) Statistical comparison of regression parameters from subsets of fatigue test results did not prove that a small number (on the order of ten) of specimens would be adequate to characterize the fatigue behavior of asphalt concrete.

e) Fatigue data can be accurately represented by energy density fatigue curves as well as stress and strain fatigue curves.

4. Cement Stabilization Test Results

Laboratory flexural fatigue tests were performed on two soil-cement mixtures. The materials and mixtures tested were representative of those placed in test pavements at the U.S. Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi (reference 35). The testing program consisted of repeatedly loading specimens of Vicksburg lean clay stabilized with 10 percent cement and WES gravelly sand stabilized with 6 percent cement. Full-wave sine loading was used for all tests which were conducted at 70°F with a loading rate of 100 cycles per minute.

Fatigue test results of these mixtures in terms of stress-fatigue life and strain-fatigue life relationships are shown in Figures 117 and 118, respectively.

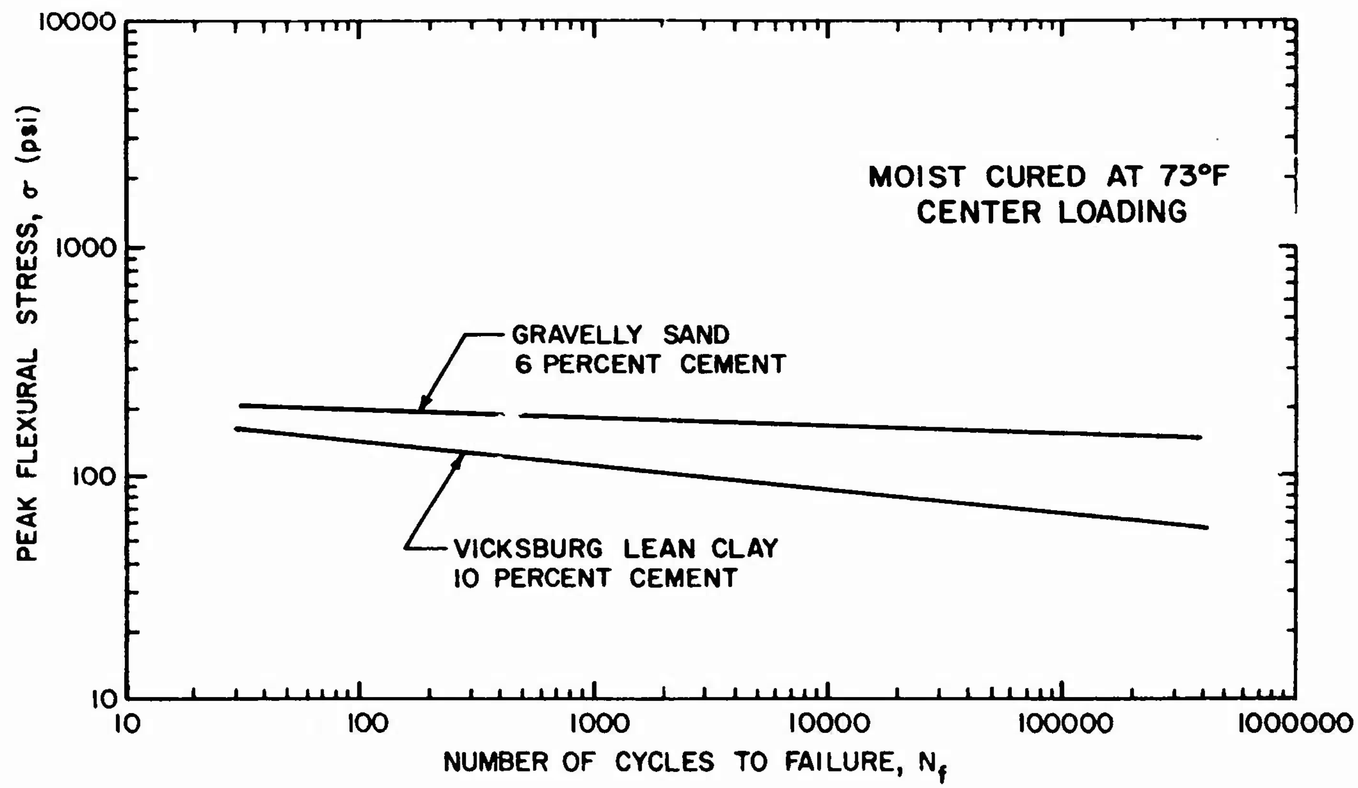


Figure 117 - Stress Fatigue Curves for Cement-Stabilized Soils

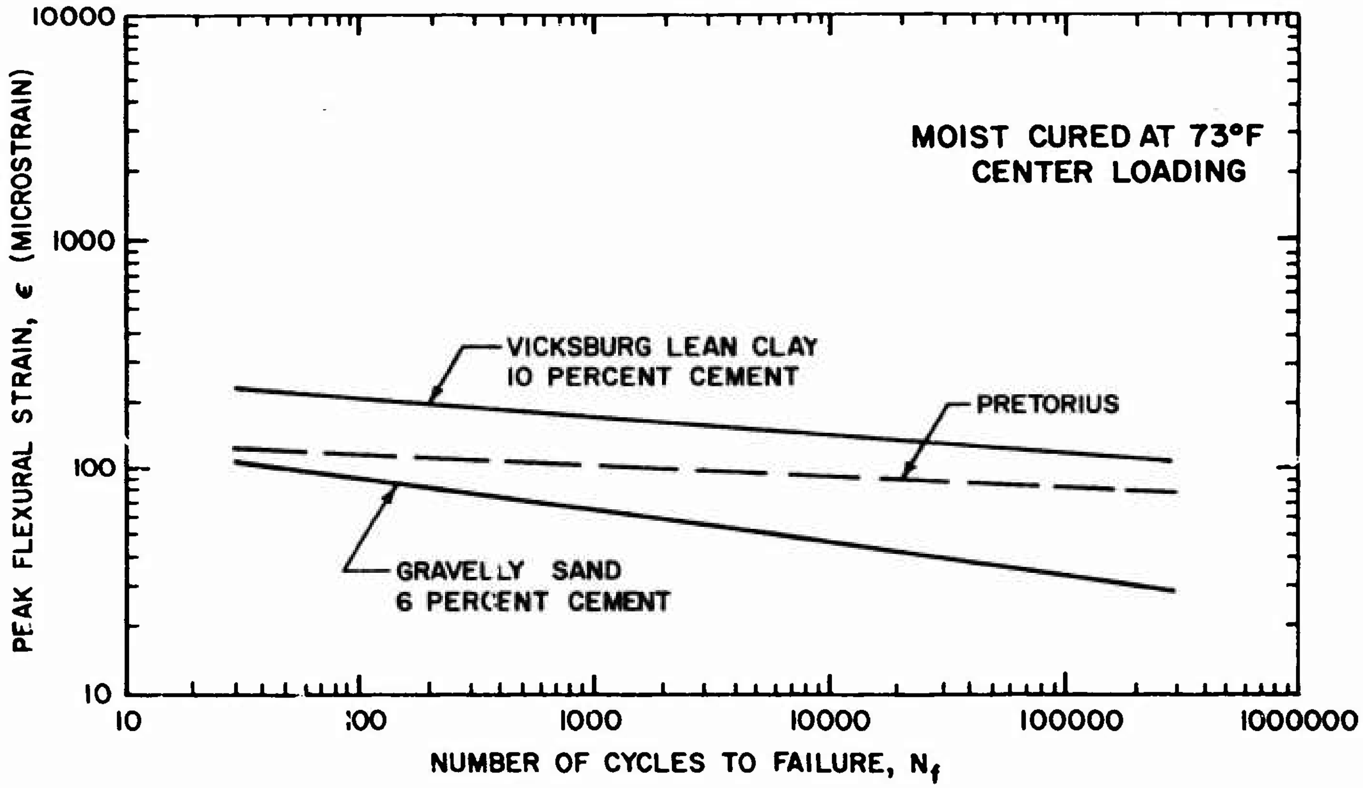


Figure 118 - Strain Fatigue Curves for Cement-Stabilized Soils

Test results by Pretorius (reference 34) are included on Figure 118 for comparison.

Resilient modulus of elasticity, density, void ratio and cube strength for the soil-cement mixtures tested are shown in Table 20.

Based upon the test results of the cement stabilization soil testing program and the extensive results presented in reference 32, the following general conclusions appear warranted.

a) The amount of time between mixing and compacting, and also the length of the curing period after compaction have a considerable influence on the strength of soil-cement. However, the fatigue behavior of materials cured for 2 weeks was not statistically different from materials cured for 4 weeks.

b) The fatigue results indicate that a fine-grained soil-cement material is preferable to a coarse-grained material from a fatigue standpoint. This is consistent with fracture mechanics theory.

c) Of the order of 15 to 20 specimens are needed to accurately characterize the fatigue behavior of soil-cement.

d) Center point loading of beam specimens is preferred to third point loading for the more brittle stabilized materials.

5. Lime Stabilization Test Results

Two series of laboratory flexural fatigue tests were performed on a single soil-lime mixture. The mixture tested was representative of the soil-lime mixture placed in test pavements at the Waterways Experiment Station as described in reference 35. The testing program consisted of repeatedly loading beam specimens of the stabilized soil after the specimens had undergone various curing conditions. Both third and center point load application were utilized

TABLE 20

AVERAGE RESILIENT MODULUS OF ELASTICITY, AVERAGE DENSITY, AVERAGE VOID RATIO, AND AVERAGE CUBE STRENGTH FOR CEMENT-STABILIZED MATERIALS

Material and Test Method	Average Resilient Modulus of Elasticity, psi	Average Density, lb./cu.ft.	Average Void Ratio	Average Cube Strength, psi
Lean Clay Third-Point Loaded	477,000	107.3	0.59	544
Lean Clay Center Loaded	621,000	108.3	0.58	530
Gravelly Sand Center Loaded	3,582,000	133.6	0.28	1078

together with full-wave sine load applied at 100 cycles per minute at a testing temperature of 70°F.

Fatigue test results are shown in Figures 119 and 120. Accelerated curing as indicated on these figures refers to a curing period of 30 hours at 120°F followed by storage at 50°F for a selected period of time. Moist curing was from 28 to 37 days at 73°F. Comparisons of test results (reference 32) indicate that the accelerated cure period is equivalent to 28 days of curing at 73°F.

Resilient modulus of elasticity, density, void ratio, degree of saturation and cube strength for the lime stabilized soils are shown in Table 21.

Based upon the test results of the lime stabilized soil testing program and the extensive results presented in reference 32, the following general conclusions appear warranted:

a) Soil-lime mixtures undergo a steady increase in strength with time.

b) Because of the weak and brittle nature of many soil-lime mixtures, it appears that 30 to 40 specimens may be required to accurately characterize the fatigue behavior of soil-lime.

6. Conclusions

Center point and third point beam specimens and plate specimens made with bituminous-, cement- and lime-stabilized soils were tested with half-wave sine and full-wave sine loading. In general, it was found preferable to utilize center point beam specimens for the stabilized soils. This was based on ease of testing and less data scatter.

Considerable difficulty was experienced in the testing of the cement- and lime-stabilized soils. However, it is felt that this type of information must be obtained in the future if pavement designers are to realistically determine

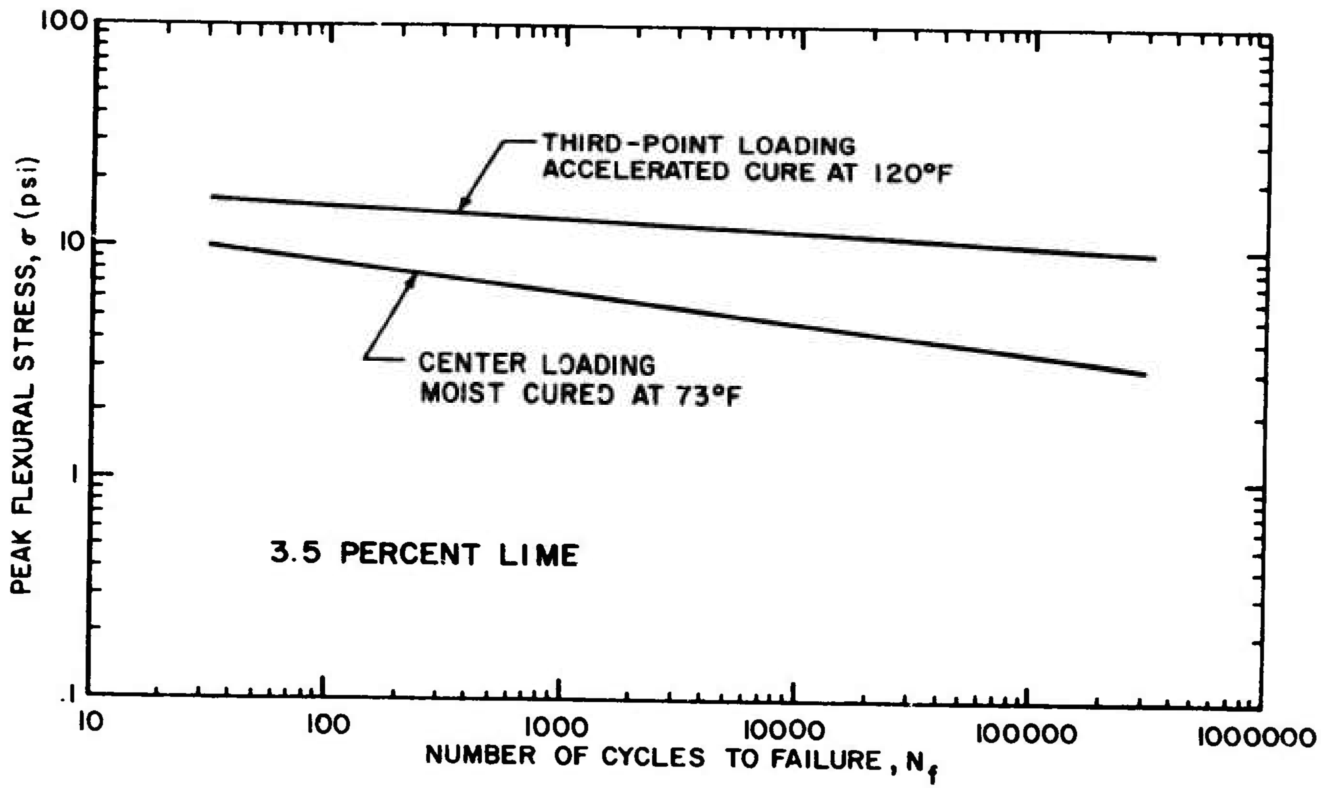


Figure 119 - Stress Fatigue Curves for Lime-Stabilized Lean Clay

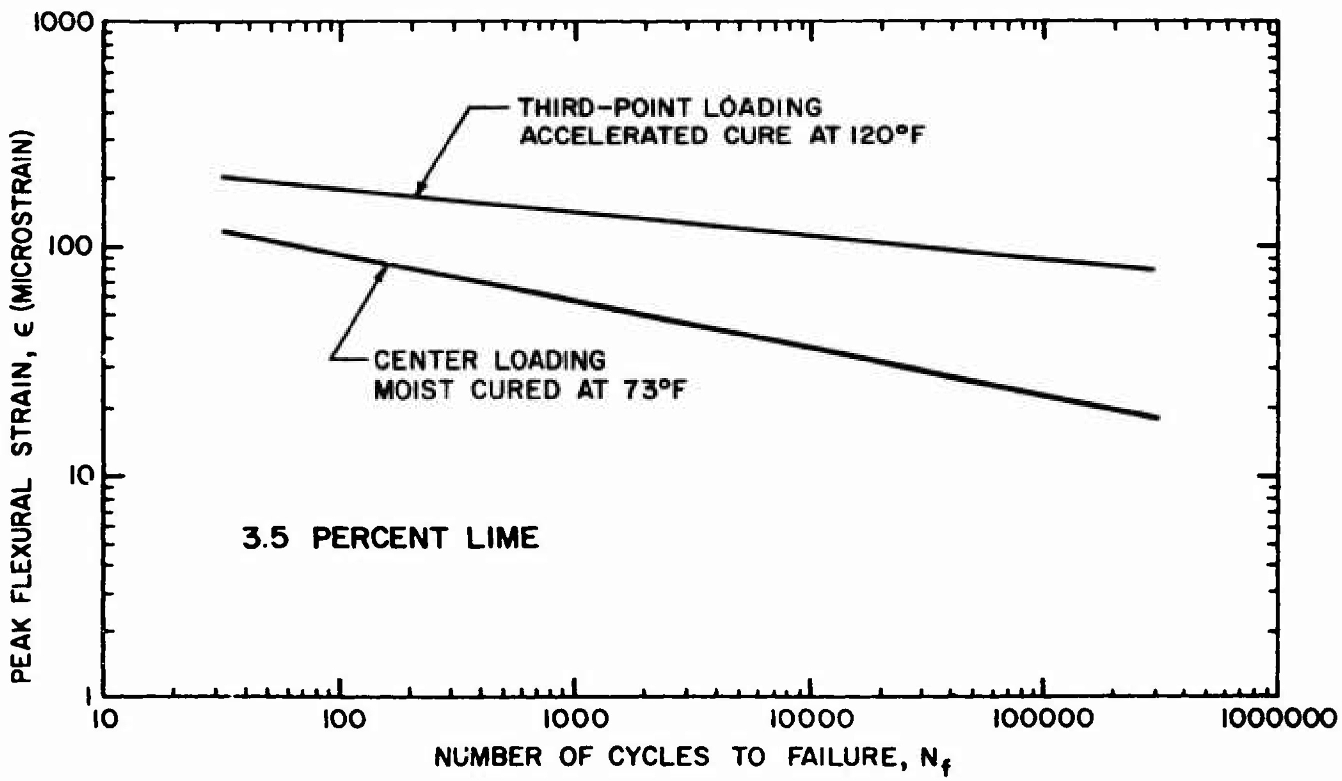


Figure 120 - Strain Fatigue Curves for Lime-Stabilized Lean Clay

TABLE 21

AVERAGE RESILIENT MODULUS OF ELASTICITY, AVERAGE DENSITY, AVERAGE VOID RATIO, AVERAGE DEGREE OF SATURATION, AND AVERAGE CUBE STRENGTH FOR LIME-STABILIZED MATERIALS

Test Series	Average Resilient Modulus of Elasticity, psi	Average Density lb./cu.ft.	Average Void Ratio	Average Degree of Saturation, Percent	Average Cube Strength, psi
Accelerated Cure Held 90-100 days at 50°F before test	154,000	104.3	0.64	75.2	122.8
Accelerated Cure Held 7-21 days at 50°F before test	94,000	103.9	0.64	74.4	117.0
Moist Cured	86,000	100.8	0.70	72.0	76.0

the structural benefits of stabilized materials. The prediction of elastic constants and failure criteria from soil and stabilizer properties are available only in very general terms at the present.

SECTION VII

SUMMARY

1. Lime Stabilization Subsystem

a. The subsystems as initially developed for lime stabilization were considered to be vague in some instances, they were time consuming, and they were not clear regarding durability requirements. On the basis of testing performed during the validation phase of the study, it appeared that changes should be made in the lime stabilization subsystems to utilize advantages gained from accelerated curing and accelerated durability testing.

b. The lime requirement obtained from strength tests (lime content at peak strength), from the pH tests and from the Atterberg limits (lime fixation) showed good agreement with each other in all but very unusual soils. For initial estimates of the lime content, either the pH test or fixation point could be used, however, the pH test is recommended. This does require specialized equipment, a pH meter. If pH meters are not available, then the lime-fixation percentage could be determined. If neither method was possible under certain expedient situations, the lime quantity would have to be estimated from available tables which are based on experience. The latter method is obviously crude and subject to error.

c. Previous research has indicated that a soil can be considered reactive with lime if the lime produces a strength increase of 50 psi after a satisfactory curing period, usually considered as 28 days at 72°F. However, the curing can be accelerated at elevated temperatures, In expedient situations, particularly for base courses, the accelerated

curing procedure is recommended. It is not yet recommended for nonexpedient situations solely because it was based on a limited number of soils. Additional research should be accomplished on a large number of soils before it is considered for nonexpedient situations. If a lime stabilized soil has an unconfined compressive strength of 110 psi after either normal or accelerated curing, it is probably lime reactive. Utilization of this fact can eliminate the testing of the untreated soil to determine its strength in expedient situations, but for nonexpedient situations the actual increase in strength should still be determined.

d. The effect of significant quantities of sulfates and organics on lime stabilization was studied. Although soils containing sulfate had high compressive strengths when they were cured at constant moisture content, the soils suffered considerable loss in strength when their moisture content was increased by immersion in water or freezing and thawing. The test results indicated that a sulfate content in excess of about 0.75 percent would seriously interfere with lime stabilization of the soil. It was not possible to determine the allowable organic content that would not interfere with lime stabilization of the soil. While it is of considerable academic interest to know the exact harmful quantities of sulfates and organics, the difficult and nonstandard nature of the tests for determining them makes it doubtful that they will be utilized in practical situations. Thus, it appears that the most suitable way of evaluating the effect of sulfates and organics - as well as other harmful substances - is through the use of laboratory durability tests. Of the three types of

durability tests which were utilized, i.e., freeze-thaw, long-term and short-term immersion, the least successful appeared to be the freeze-thaw test: the lime stabilized soils appeared to have little resistance to freezing and thawing. Fortunately, there appeared to be a correlation between the various durability tests. The correlation between immersed strengths and freeze-thaw durability shows promise in predicting freeze-thaw durability of soil-lime mixtures just by knowing the immersed strengths. A correlation was also found between short-term and long-term immersion durability. It was noted that the initial unconfined compressive strength did not correlate with any of the durability tests, thus ruling this out as a possible quick means of evaluating durability. The short-term immersion test (1-day immersion after curing) could be utilized for expedient situations, particularly for base courses, if the specimens were cured using the rapid curing procedures.

e. For the soils tested, it was observed that for all soil-lime mixtures except one (the Tyler soil) the addition of lime in excess of that required to achieve maximum 28-day strength had no beneficial effect on the immersion or freeze-thaw durability.

2. Cement Stabilization Subsystem

a. The original subsystems for cement stabilization were based on more complete data than those for lime, and extensive revision was not necessary. The primary change involved was adding the pH test for harmful organic matter into the expedient subsystems. The pH criterion was changed from 12.0 to 12.1 based on experimental results on a limited number of soils.

b. In general, it appeared that sulfate contents below 1 percent did not have a detrimental effect on the strength and durability of soil-cement mixtures. Soils containing less than about 1.4 percent organic matter were stabilized with cement and became durable; however the allowable amount of organic matter may be governed by the kind of organic matter and the type of soil.

c. The PCA procedure for determining estimated cement requirements appears to be a valid approach only for soils containing little or no sulfate and/or organic matter.

d. The long-term immersion test appears to be a promising method for predicting freeze-thaw and wet-dry durability of soil-cement mixtures.

3. Bituminous Stabilization Subsystem

a. The gradation and plasticity limits originally set in the bituminous stabilization subsystems were designed to insure adequate mixing of the bitumen and soil. Simple laboratory mixing experiments on many different soils verified these limits.

b. Changes have been made in the bituminous stabilization subsystems to utilize new test temperatures for Marshall specimens when determining suitability of stabilized materials. The test temperatures depend on the location of the stabilized material in the pavement structure and the environment. However, for selection of the asphalt quantity, a testing temperature of 140°F is still utilized.

c. Until better criteria become available, design criteria for mixtures containing liquid asphalts should be identical to those for asphalt cement mixtures.

SECTION VIII

RECOMMENDATIONS

The Soil Stabilization Index System presented herein represents only one of many approaches which could have been taken to satisfy the project objectives. The System has evolved and now bears little resemblance to the first attempt by the authors to systematize soil stabilization. At all stages of development, however, the philosophy of pragmatism has prevailed. The need for basic research in soil stabilization, which the Air Force has also sponsored, need not be defended. On the other hand, much of this information cannot be directly applied in the field by the engineer charged with the task of determining whether to stabilize and what stabilizer to use. The complicated nature of the tests and techniques used by the researcher is one reason why this is true. With few exceptions, the tests used in SSIS are of an engineering nature and can be performed with relative ease in the field. It is believed that this concept should continue and the results of basic research should be implemented into SSIS only after the appropriate tests and interpretations have been developed commensurate with the capabilities of the field engineering forces.

There are undoubtedly many improvements to be made within the framework of the present SSIS, and in this respect it must be considered dynamic rather than static: it is to be expected that changes will be made in the System. The recommendations below are in line with this philosophy.

a. SSIS was based on the combined experience of many individuals as reported in the literature. Very seldom did these individuals have at hand and report all of the soils information called for in SSIS, and only a limited number of soils could be tested in the validation phase of the project. As a result, SSIS has been used on relatively

few soils. The first and foremost task should be to examine many soils using SSIS. It is difficult to suggest numbers, but well over 100 soils of widely varying properties should be examined. Soils with field records of stabilization should be utilized as much as possible.

b. Work on accelerated curing of lime stabilized samples should be continued. The present work has been conducted on soils of widely varying properties, but, again, only a few soils have been utilized.

c. Investigation of the effects of organics and sulfates on both lime- and cement-stabilized soils should be continued. Controlled addition of sulfates in the laboratory may be feasible, but it is felt that naturally occurring organic soils must be utilized.

d. Attempts should be made to develop a simple test for the determination of sulfates which can be utilized in the field.

e. Development of durability criteria for stabilized soils should be continued. Laboratory tests bear little resemblance to field conditions and the appropriate criteria should be based on both laboratory study and field performance of stabilized soils.

f. New attempts should be made to develop accelerated curing techniques for cement stabilized soils. The pick-and-click test is not very quantitative, and accelerated curing at higher temperatures could lead to elimination of the personal judgment associated with the present approach.

g. Immediate steps should be taken to supply suitable pH meters to Air Force construction units which will be utilizing SSIS.

APPENDIX A

EXPEDIENT SUBGRADE STABILIZATION SYSTEM

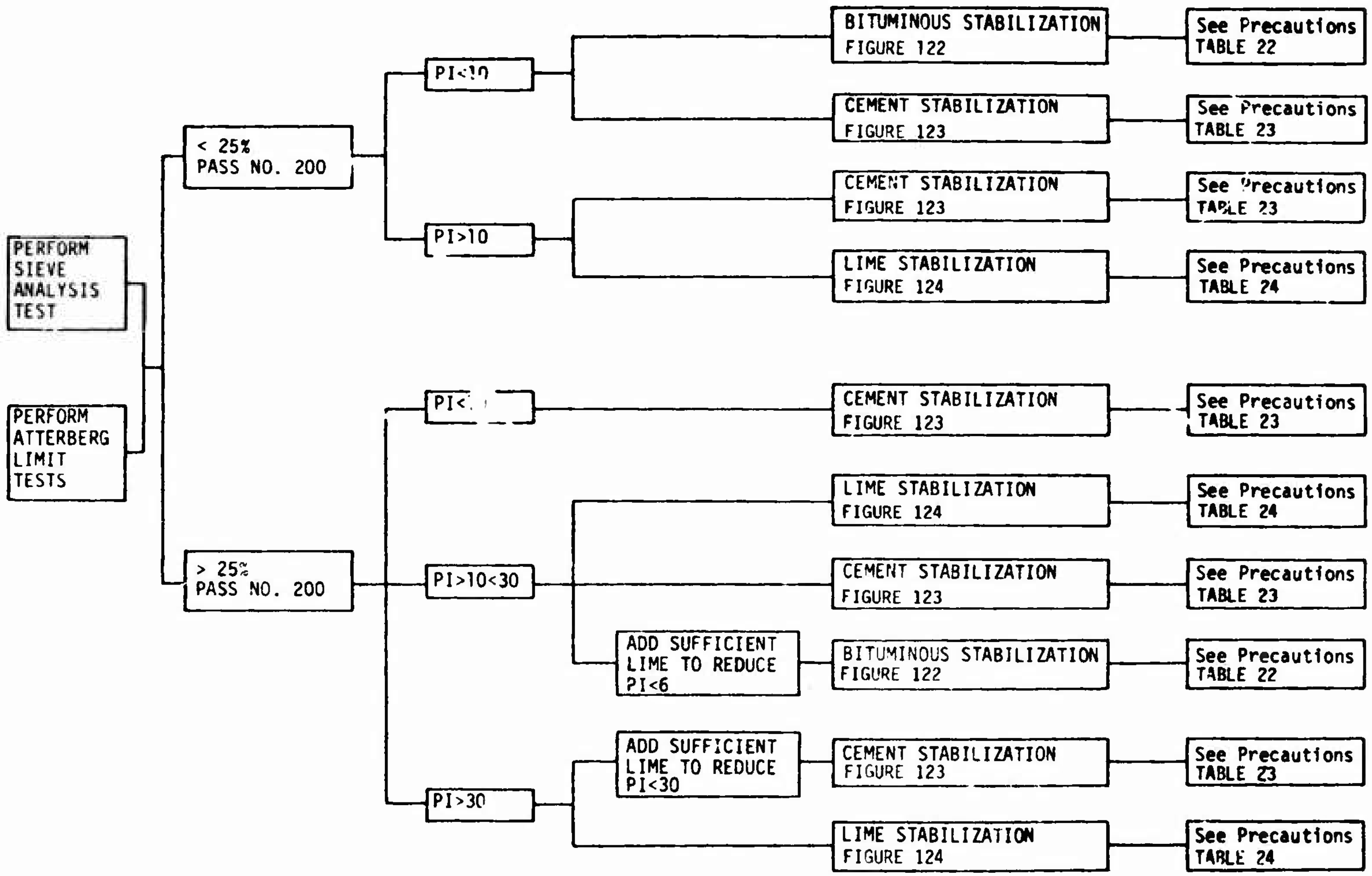


Figure 121 - Selection of Stabilizer for Expedient Subgrade Construction

TABLE 22

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR BITUMINOUS STABILIZATION IN EXPEDIENT SUBGRADES

Condition	Precautions
Environmental	When cutbacks and emulsions are utilized, the air temperature and soil temperature should be above freezing. Bituminous materials should completely coat the soil particles before rainfall stops construction.
Construction	Hot dry weather is preferred for all types of bituminous stabilization.

TABLE 23

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR CEMENT STABILIZATION IN EXPEDIENT SUBGRADES

Condition	Precautions
Environmental	If the soil temperature is less than 40°F and is not expected to increase for one month, chemical reactions will not occur rapidly, and strength gain of the cement-soil mixture will be minimal. If these environmental conditions are expected the cement may act as a modifier.
Construction	If heavy vehicles are allowed on the cement stabilized soils prior to a 10 to 14 day curing period, certain pavement damage can be expected. Construction during periods of heavy rainfall should be avoided. Compaction of cement stabilized soil should be completed within 5 to 6 hours after spreading and mixing.

TABLE 24

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR LIME STABILIZATION IN EXPEDIENT SUBGRADES

Condition	Precautions
Environmental	If the soil temperature is less than 40°F and is not expected to increase for one month, chemical reactions will not occur rapidly, and strength gain of the cement-soil mixture will be minimal. If these environmental conditions are expected the cement may act as a modifier.
Construction	If heavy vehicles are allowed on the cement stabilized soils prior to a 10 to 14 day curing period, certain pavement damage can be expected. Construction during periods of heavy rainfall should be avoided. Compaction of cement stabilized soil should be completed within 5 to 6 hours after spreading and mixing.

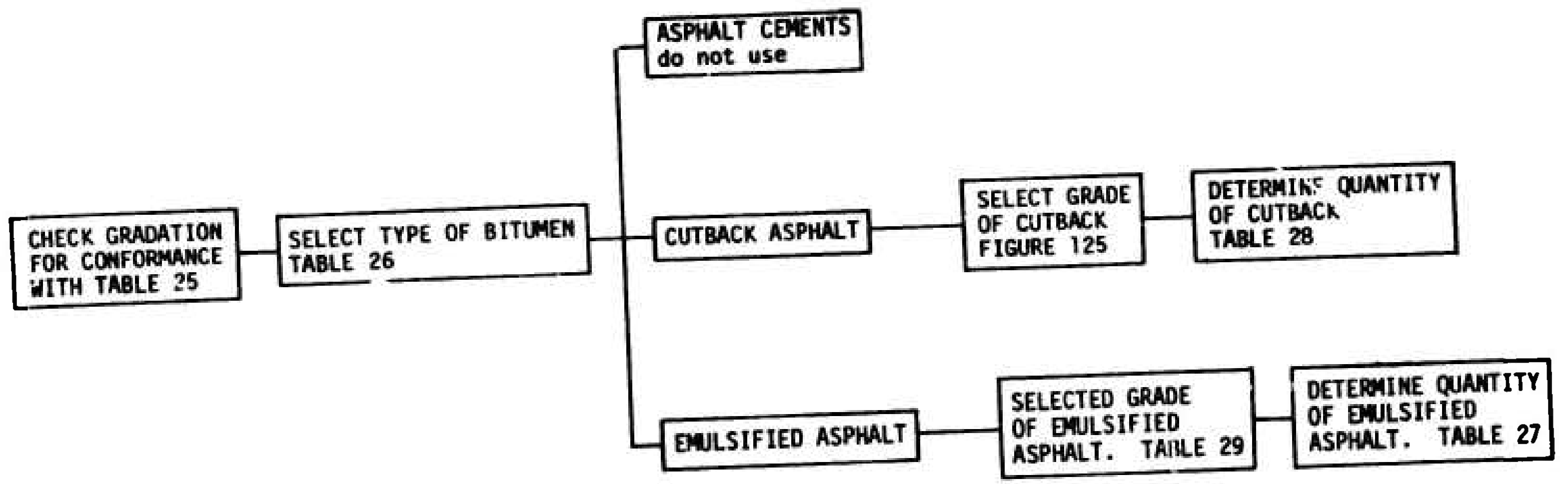


Figure 122 - Subsystem for Expedient Subgrade Stabilization with Bituminous Materials

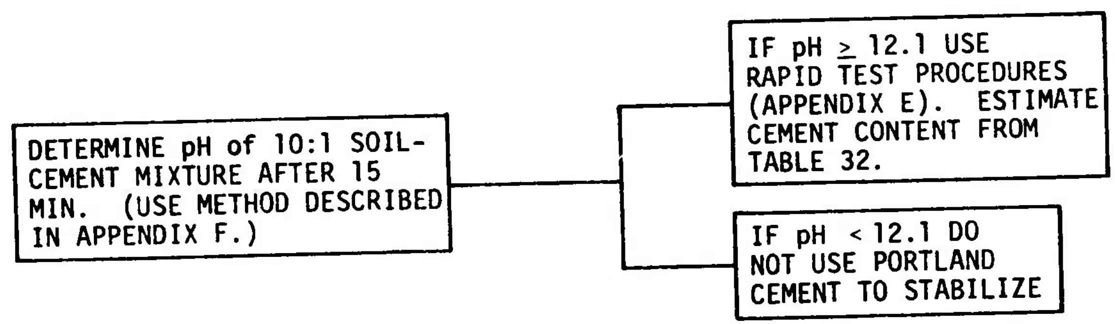


Figure 123 - Subsystem for Expedient Subgrade Stabilization with Cement

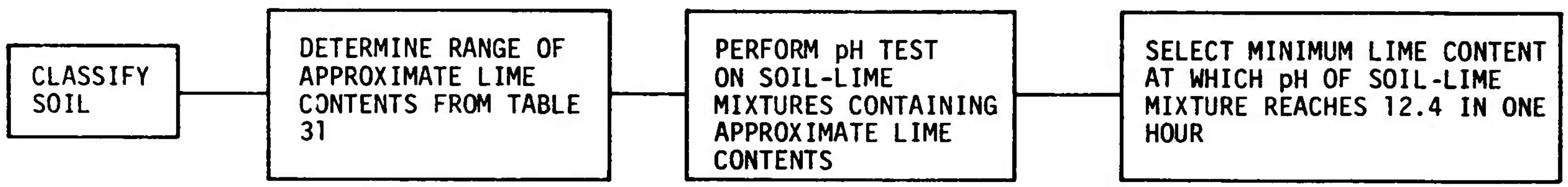
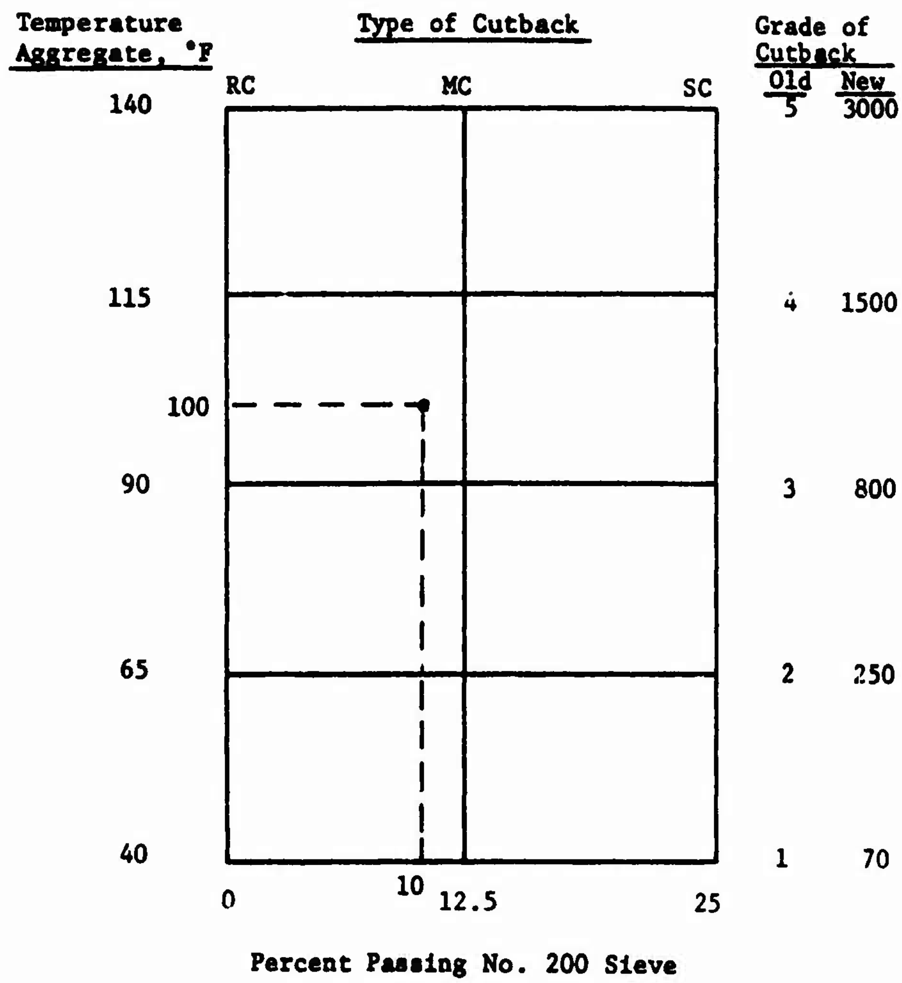


Figure 124 - Subsystem for Expedient Subgrade Stabilization with Lime

NOTE: If adequate time is available, the procedure for expedient base course should be used.



Example: For aggregate temperature of 100°F and 10 percent passing No. 200 sieve, use MC 800 cutback.

Figure 125 - Selection of Type of Cutback for Stabilization

(from reference 38)

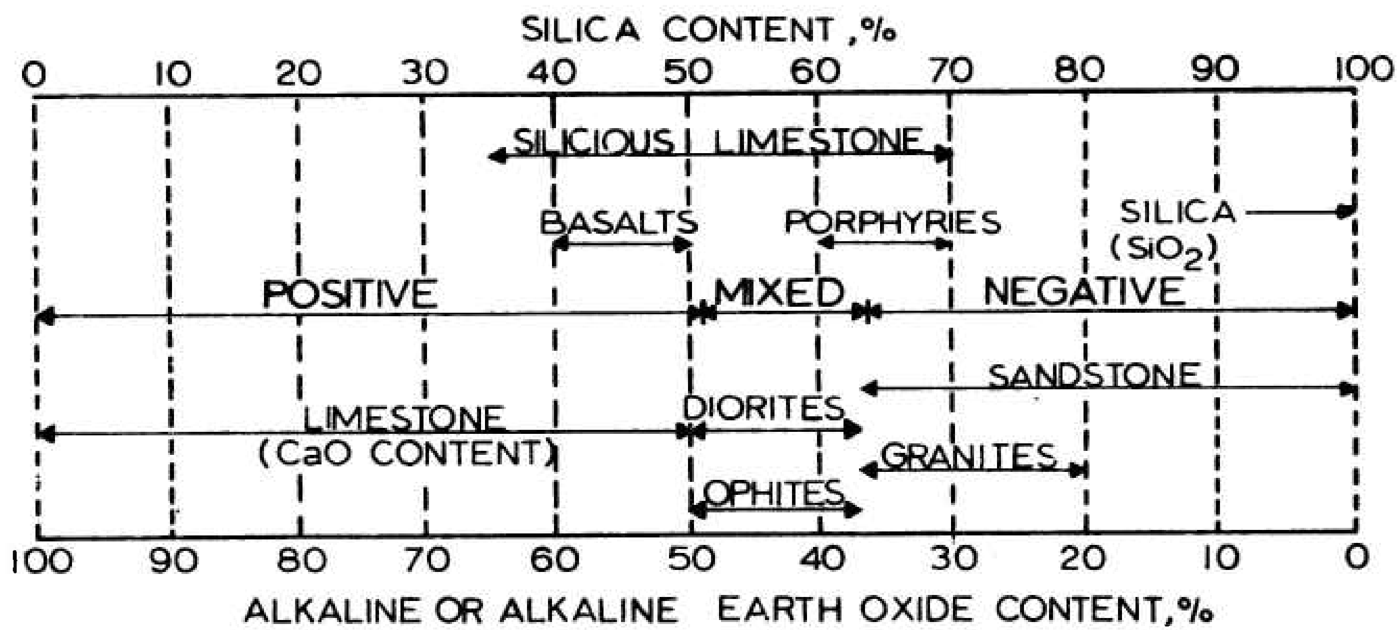


Figure 126 - Classification of Aggregates

(from reference 39)

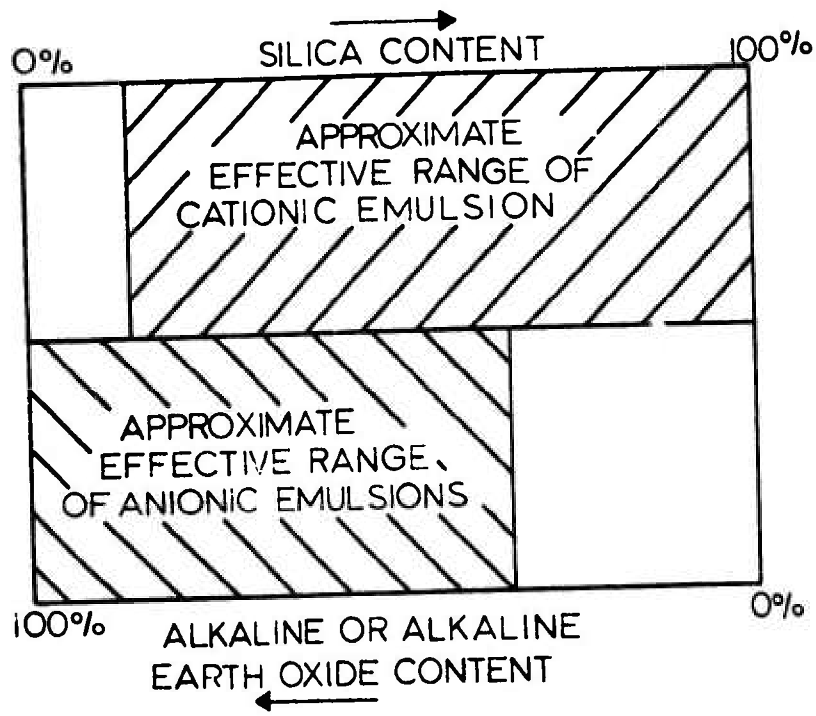


Figure 127 - Approximate Effective Range of Cationic and Anionic Emulsions on Various Types of Aggregates

(from reference 39)

TABLE 25

ENGINEERING PROPERTIES OF MATERIALS

SUITABLE FOR BITUMINOUS STABILIZATION

Percent Passing Sieve	Sand-Bitumen	Soil-Bitumen	Sand-Gravel-Bitumen
1-1/2 inch			100
1 inch	100		
3/4 inch			60-100
No. 4	50-100	50-100	35-100
10	40-100		
40		35-100	13-50
100			8-35
200	5-12	good - 3-20 fair - 0-3 and 20-25	0-12

Includes slight modifications later made by Herrin.

(from reference 40)

TABLE 26

SELECTION OF A SUITABLE TYPE OF BITUMEN

FOR SOIL STABILIZATION PURPOSES

Sand Bitumen	Soil Bitumen	Crushed Stones and Sand-Gravel Bitumen
Cold Mix: Cutbacks See Figure 125 Elumsions See Table 29 See Figures 126 and 127 to determine if a cationic or anionic emulsion should be used	Cold Mix: Cutbacks See Figure 125 Emulsions See Table 29 See Figures 126 and 127 to determine if a cationic or anionic emulsion should be used	Cold Mix: Cutbacks See Figure 125 Emulsions See Table 29 See Figures 126 and 127 to determine if a cationic or anionic emulsion should be used

TABLE 27

EMULSIFIED ASPHALT REQUIREMENT

Percent passing No. 200	Lbs. of emulsified asphalt per 100 lbs. of dry aggregate when percent passing No. 10 sieve is:					
	50*	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

*50 or less.

(from reference 38)

TABLE 28

DETERMINATION OF QUANTITY OF CUTBACK ASPHALT

$p = 0.02\,(a) + 0.07\,(b) + 0.15\,(c) + 0.20\,(d)$	
Symbol	Definition
p	percent of residual asphalt by weight of dry aggregate*
a	percent of mineral aggregate retained on No. 50 sieve
b	percent of mineral aggregate passing No. 50 and retained on No. 100 sieve
c	percent of mineral aggregate passing No. 100 and retained on No. 200 sieve
d	percent of mineral aggregate passing No. 200 sieve

*Percent cutback can be obtained by referring to Table 30 and utilizing the following equation:

$$\text{percent cutback} = \frac{\text{percent residual asphalt (p)}}{(100 - \text{percent solvent})} \times 100$$

TABLE 29

SELECTION OF TYPE OF EMULSIFIED ASPHALT

FOR STABILIZATION

Percent Passing No. 200 Sieve	Relative Water Content of Soil	
	Wet (5 percent +)	Dry (0-5 percent)
0-5	SS-1h (CSS-1h)	CMS-2h (or SS-1h*)
5-15	SS-1, SS-1h (CSS-1, CSS-1h)	CMS-2h (or SS-1h*, SS-1*)
15-25	SS-1 (CSS-1)	CMS-2h

*Soil should be pre-wetted with water before using these types of emulsified asphalts.

(from reference 38)

TABLE 30

ASPHALT CUTBACK COMPOSITION

Type of Cutback	Solvent	Percent Solvent for Particular Grades				
		30	70	250	800	3000
RC	Gasoline or Naptha	--	35	25	17	13
MC	Kerosene	46	36	26	19	14
SC	Fuel Oil	--	50	40	30	20

TABLE 31

APPROXIMATE LIME CONTENTS

Soil Type	Approximate Treatment, Percent by Soil Weight	
	Hydrated Lime	Quicklime
Clayey Gravels (GC, GM-GC) (A-2-6, A-2-7)	2-4	2-3
Silty Clays (CL) (A-6, A-7-6)	5-10	3-8
Clays (CH) (A-6, A-7-6)	3-8	3-6

(from reference 41)

TABLE 32

CEMENT REQUIREMENTS FOR VARIOUS SOILS

AASHO Soil Classification	Unified Soil Classification*	Usual Range in cement requirement**		Estimated cement content and that used in moisture-density test, percent by weight	Cement contents for wet-dry and freeze-thaw tests, percent by weight
		percent by vol.	percent by wt.		
A-1-a	GW, GP, GM, SW, SP, SM	5 - 7	3 - 5	5	3- 5- 7
A-1-b	GM, GP, SM, SP	7 - 9	5 - 8	6	4- 6- 8
A-2	GM, GC, SM, SC	7 - 10	5 - 9	7	5- 7- 9
A-3	SP	8 - 12	7 - 11	9	7- 9-11
A-4	CL, ML	8 - 12	7 - 12	10	8-10-12
A-5	ML, MH, CH	8 - 12	8 - 13	10	8-10-12
A-6	CL, CH	10 - 14	9 - 15	12	10-12-14
A-7	OH, MH, CH	10 - 14	10 - 16	13	11-13-15

*based on correlation presented by Air Force
**for most A horizon soils the cement should be increased 4 percentage points, if the soil is dark grey to grey, and 6 percentage points if the soil is black.

(from reference 24)

APPENDIX B

EXPEDIENT BASE COURSE STABILIZATION SYSTEM

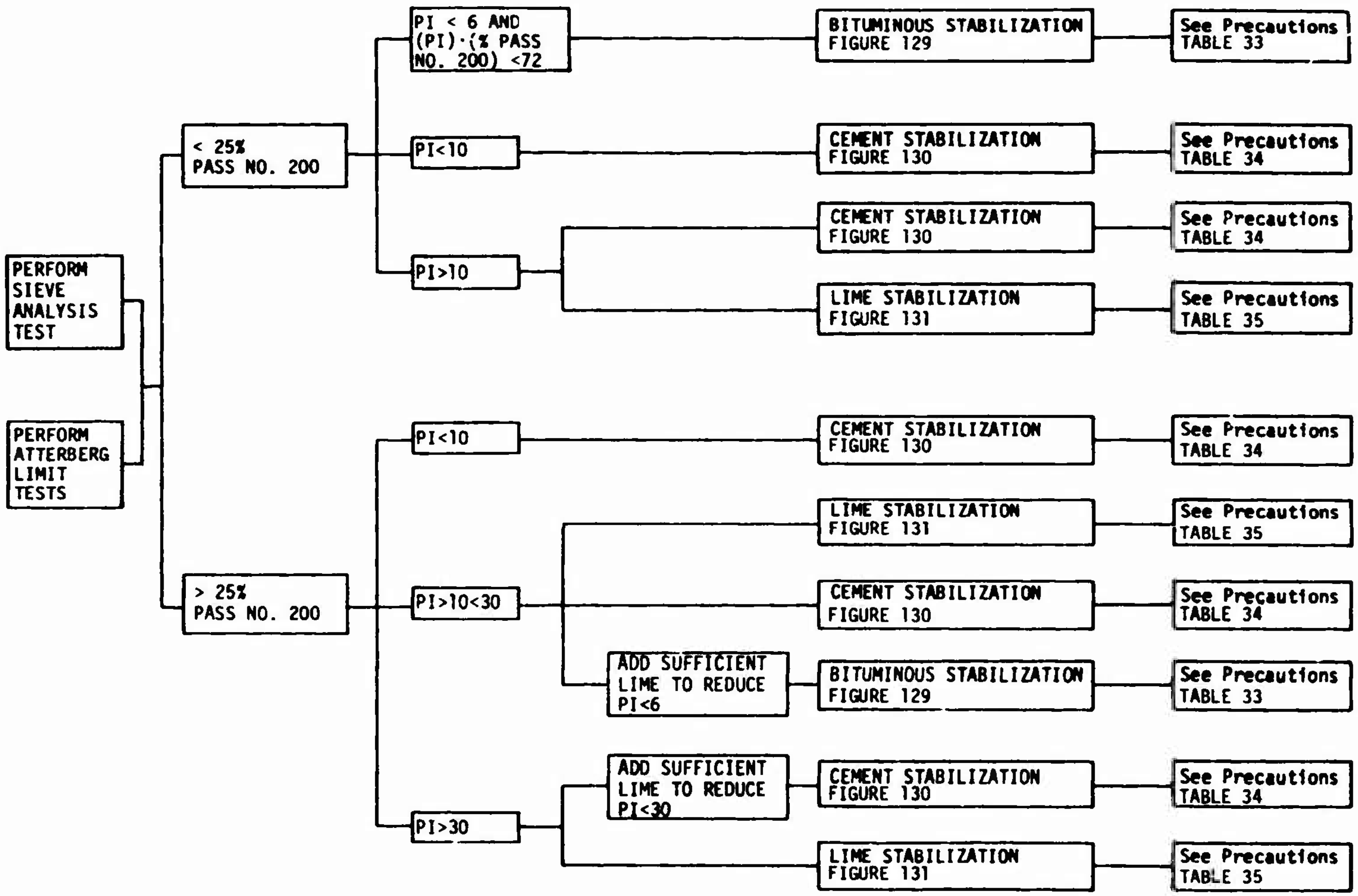


Figure 128 - Selection of Stabilizer for Expedient Base Construction

TABLE 33

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR BITUMINOUS STABILIZATION IN EXPEDIENT BASE COURSES

Condition	Precautions
Environmental	When asphalt cements are used for bituminous stabilization, proper compaction must be obtained. If thin lifts of asphalt concrete are being placed, the air temperature should be 40°F and rising, and compaction equipment should be used immediately after lay down operation. Adequate compaction can be obtained at freezing temperatures if thick lifts are utilized. When cutbacks and emulsions are utilized, the air temperature and soil temperature should be above freezing. Bituminous materials should completely coat the soil particles before rainfall stops construction.
Construction	Central batch plants together with other specialized equipment, are necessary for bituminous stabilization with asphalt cements. Hot dry weather is preferred for all types of bituminous stabilization.

TABLE 34

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR CEMENT STABILIZATION IN EXPEDIENT BASE COURSES

Condition	Precautions
Environmental	If the soil temperature is less than 60 to 70°F and is not expected to increase for one month, chemical reactions will not occur rapidly, and strength gain of the cement-soil mixture will be minimal. If these environmental conditions are expected, an alternative stabilizer should be investigated for possible use.
Construction	If heavy vehicles are allowed on the cement stabilized soils prior to a 10 to 14 day curing period, certain pavement damage can be expected.

TABLE 35

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR LIME STABILIZATION IN EXPEDIENT BASE COURSES

Condition	Precautions
Environmental	If the soil temperature is less than 60 to 70°F and is not expected to increase for one month, chemical reactions will not occur rapidly, and the strength gain of the lime-soil mixture will be minimal. If these environmental conditions are expected an alternative stabilizer should be investigated for possible use.
Construction	If heavy vehicles are allowed on the lime stabilized soils prior to a 10 to 14 day curing period, certain pavement damage can be expected.

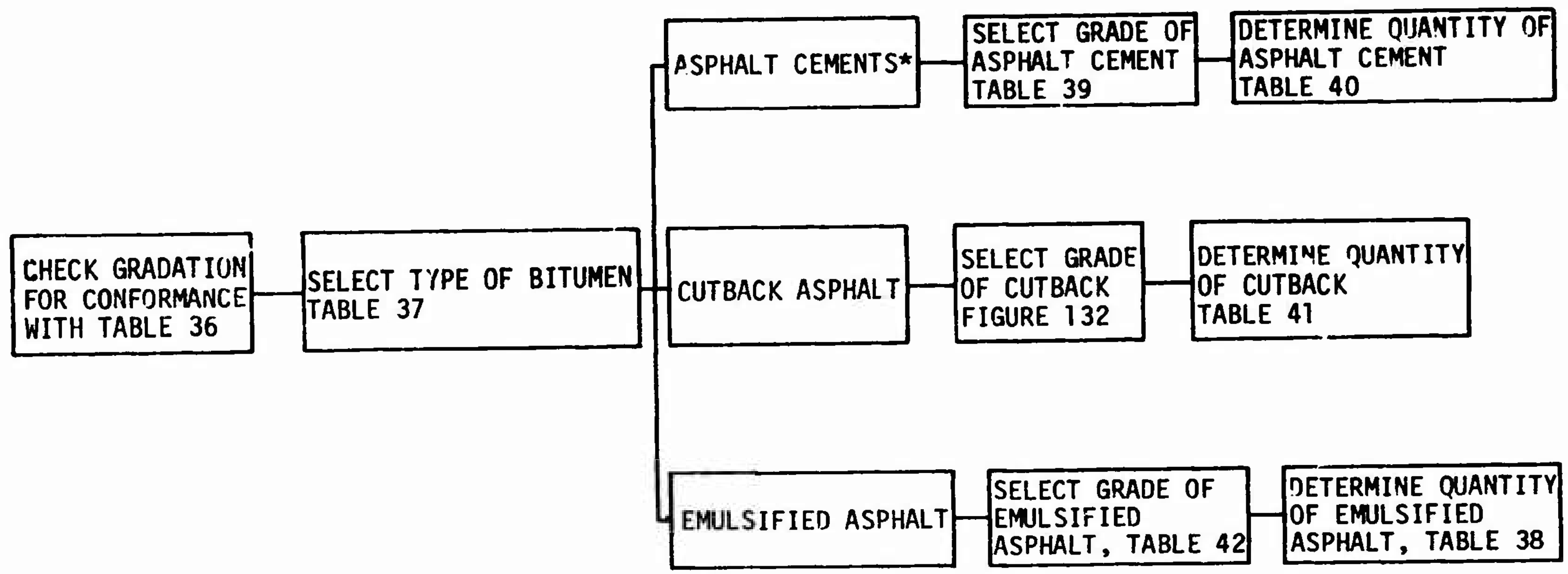


Figure 129 - Subsystem for Expedient Base Course Stabilization with Bituminous Materials

*Hard asphalt cements are preferred in hot climates.

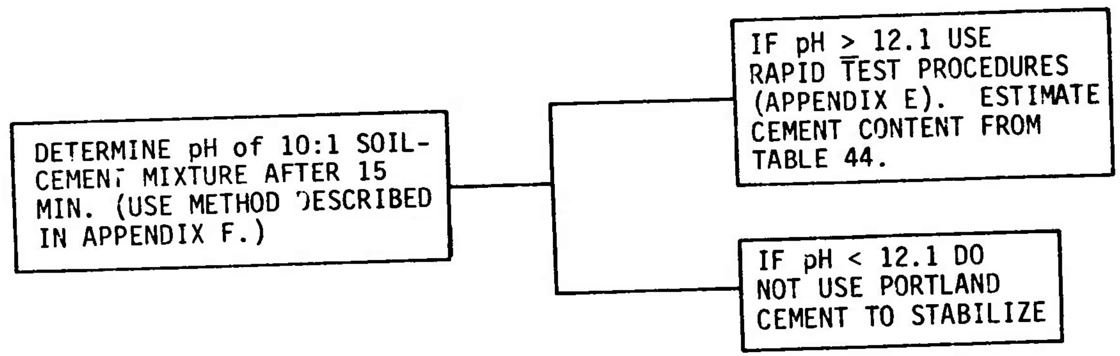


Figure 130 - Subsystem for Expedient Base Course Stabilization with Cement

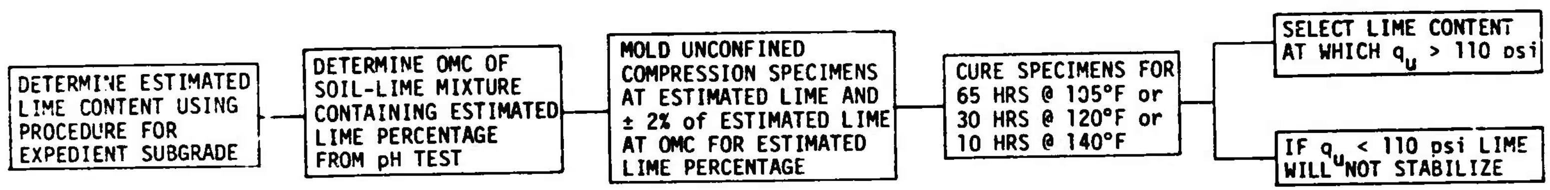
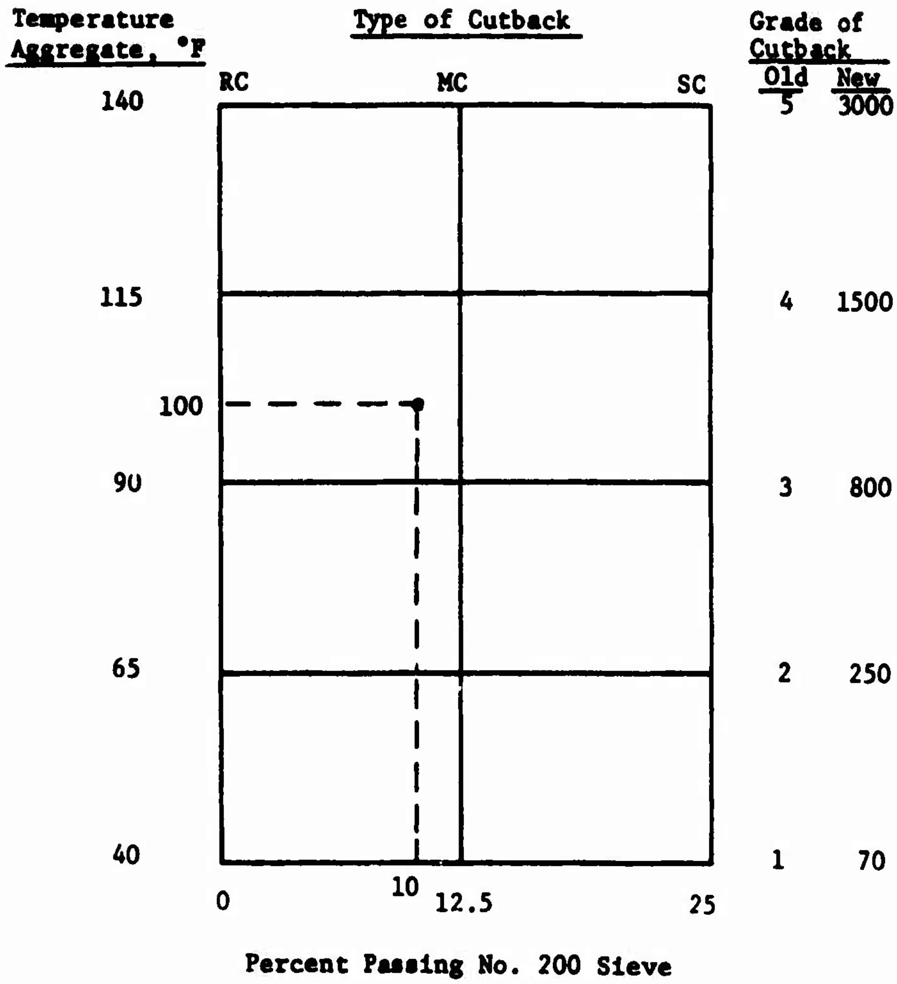


Figure 131 - Subsystem for Expedient Base Course Stabilization with Lime



Example: For aggregate temperature of 100°F and 10 percent passing No. 200 sieve, use MC 800 cutback.

Figure 132 - Selection of Type of Cutback for Stabilization

(from reference 38)

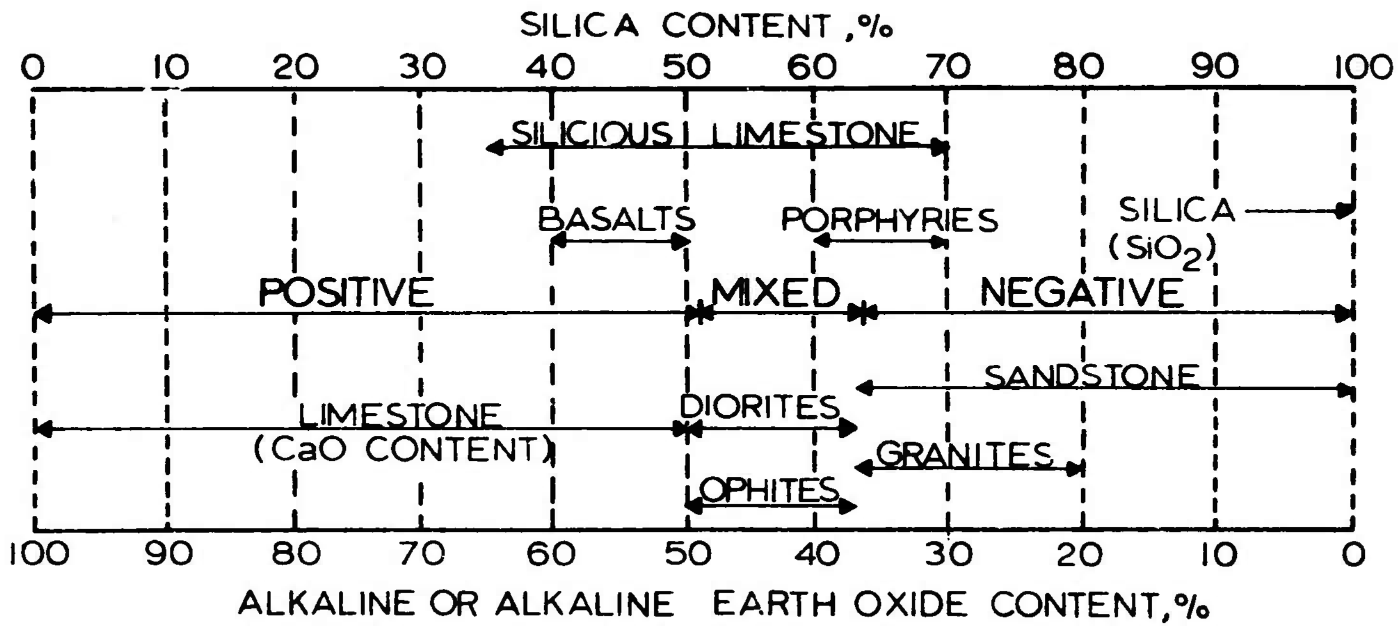


Figure 133 - Classification of Aggregates

(from reference 39)

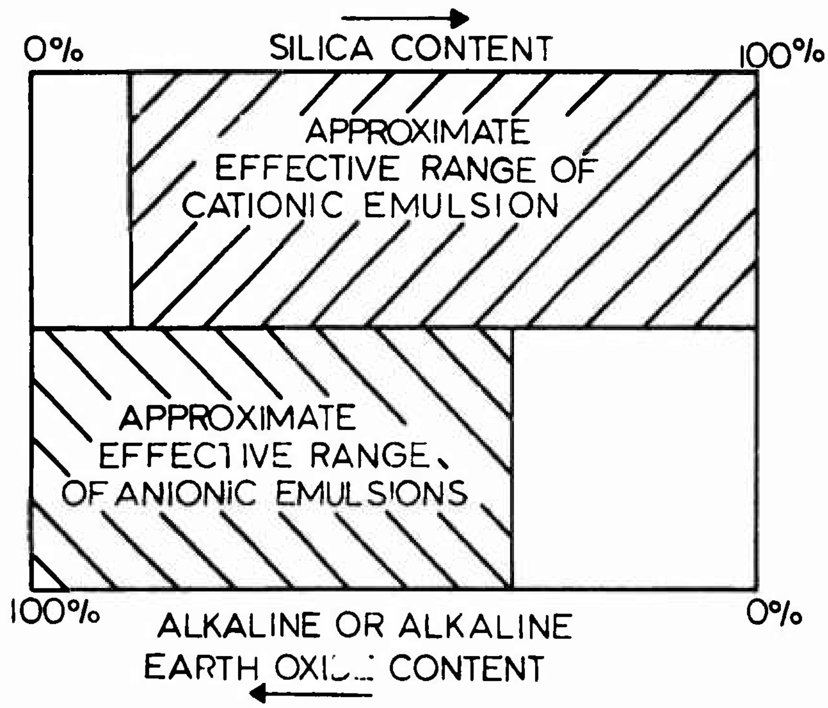


Figure 134 - Approximate Effective Range of Cationic and Anionic Emulsions on Various Types of Aggregates

(from reference 39)

TABLE 36

ENGINEERING PROPERTIES OF MATERIALS

SUITABLE FOR BITUMINOUS STABILIZATION

Percent Passing Sieve	Sand-Bitumen	Soil-Bitumen	Sand-Gravel-Bitumen
1-1/2 inch			100
1 inch	100		
3/4 inch			60-100
No. 4	50-100	50-100	35-100
10	40-100		
40		35-100	13-[illegible]0
100			8-35
200	5-12	good - 3-20 fair - 0-3 and 20-25	0-12

Includes slight modifications later made by Herrin.

(from reference 40)

TABLE 37

SELECTION OF A SUITABLE TYPE OF BITUMEN

FOR SOIL STABILIZATION PURPOSES

Sand Bitumen	Soil Bitumen	Crushed Stones and Sand-Gravel Bitumen
Hot Mix: Asphalt Cements 60-70 hot climate 85-100 120-150 cold climate		Hot Mix: Asphalt Cements 40-50 hot climate 60-70 85-100 cold climate
Cold Mix: Cutbacks See figure 132 Emulsions See table 42 See figures 133 and 134 to determine if a a cationic or anionic emulsion should be used	Cold Mix: Cutbacks See figure 132 Emulsions See table 42 See figures 133 and 134 to determine if a cationic or anionic emulsion should be used	Cold Mix: Cutbacks See figure 132 Emulsions See table 42 See figures 133 and 134 to determine if a cationic or anionic emulsion should be used

TABLE 38

EMULSIFIED ASPHALT REQUIREMENT

Percent passing No. 200	Lbs. of emulsified asphalt per 100 lbs. of dry aggregate when percent passing No. 10 sieve is:					
	50*	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

*50 or less.

(from reference 38)

TABLE 39

DETERMINATION OF ASPHALT GRADE FOR

BASE COURSE STABILIZATION

Pavement Temperature Index*	Asphalt Grade, Penetration
Negative	100-120
0-40	85-100
40-100	60-70
Above 100	40-50

*The sum, for a 1-year period, of the increments above 75°F of monthly averages of the daily maximum temperatures. Average daily maximum temperatures for the period of record should be used where 10 or more years of record are available. For records of less than 10-year duration the record for the hottest year should be used. A negative index results when no monthly average exceeds 75°F. Negative indexes are evaluated merely by subtracting the largest monthly average from 75°F.

TABLE 40

SELECTION OF ASPHALT CEMENT CONTENT

FOR EXPEDIENT BASE COURSE CONSTRUCTION

Aggregate Shape and Surface Texture	Percent Asphalt by Weight of Dry Aggregate*
Rounded and Smooth	4
Angular and Rough	6
Intermediate	5

*Approximate quantities which may be adjusted in field based on observation of mix and engineering judgment.

TABLE 41

DETERMINATION OF QUANTITY OF CUTBACK ASPHALT

$p = 0.02\,(a) + 0.07\,(b) + 0.15\,(c) + 0.20\,(d)$	
Symbol	**Definition**
p	percent of residual asphalt by weight of dry aggregate*
a	percent of mineral aggregate retained on No. 50 sieve
b	percent of mineral aggregate passing No. 50 and retained on No. 100 sieve
c	percent of mineral aggregate passing No. 100 and retained on No. 200 sieve
d	percent of mineral aggregate passing No. 200 sieve

*Percent cutback can be obtained by referring to Table 43 and utilizing the following equation:

$$\text{percent cutback} = \frac{\text{percent residual asphalt (p)}}{(100 - \text{percent solvent})} \times 100$$

TABLE 42

SELECTION OF TYPE OF EMULSIFIED ASPHALT FOR STABILIZATION

Percent Passing No. 200 Sieve	Relative Water Content of Soil	
	Wet (5 percent +)	Dry (0-5 percent)
0-5	SS-1h (CSS-1h)	CMS-2h (or SS-1h*)
5-15	SS-1, SS-1h (CSS-1, CSS-1h)	CMS-2h (or SS-1h*, SS-1*)
15-25	SS-1 (CSS-1)	CMS-2h

*Soil should be pre-wetted with water before using these types of emulsified asphalts.

(from reference 38)

TABLE 43

ASPHALT CUTBACK COMPOSITION

Type of Cutback	Solvent	Percent Solvent for Particular Grades				
		30	70	250	800	3000
RC	Gasoline or Naptha	--	35	25	17	13
MC	Kerosene	46	36	26	19	14
SC	Fuel Oil	--	50	40	30	20

TABLE 44

CEMENT REQUIREMENTS FOR VARIOUS SOILS

AASHO Soil Classification	Unified Soil Classification*	Usual Range in cement requirement**		Estimated cement content and that used in moisture-density test, percent by weight	Cement contents for wet-dry and freeze-thaw tests, percent by weight
		percent by vol.	percent by wt.		
A-1-a	GW, GP, GM, SW, SP, SM	5 - 7	3 - 5	5	3- 5- 7
A-1-b	GM, GP, SM, SP	7 - 9	5 - 8	6	4- 6- 8
A-2	GM, GC, SM, SC	7 - 10	5 - 9	7	5- 7- 9
A-3	SP	8 - 12	7 - 11	9	7- 9-11
A-4	CL, ML	8 - 12	7 - 12	10	8-10-12
A-5	ML, MH, CH	8 - 12	8 - 13	10	8-10-12
A-6	CL, CH	10 - 14	9 - 15	12	10-12-14
A-7	OH, MH, CH	10 - 14	10 - 16	13	11-13-15

*based on correlation presented by Air Force
**for most A horizon soils the cement should be increased 4 percentage points, if the soil is dark grey to grey, and 6 percentage points if the soil is black.

(from reference 24)

TABLE 45

TENTATIVE SHORT-TERM IMMERSED STRENGTH REQUIREMENTS

FOR SOIL-LIME MIXTURES

Anticipated Use	Residual Strength Requirement, psi[a]	Short-Term Immersed Strength[b] Requirements
Modified Subgrade	20	30
Subbase		
Rigid Pavement	20	30
Flexible Pavement		
Thickness of Cover		
10 inches	30	45
8 inches	40	55
5 inches	60	80
Base	100	130

[a]As recommended by Thompson (reference 10).

[b]Unconfined compressive strength of 2- by 4-inch specimen after short-term immersion.

APPENDIX C

NONEXPEDIENT SUBGRADE STABILIZATION SYSTEM

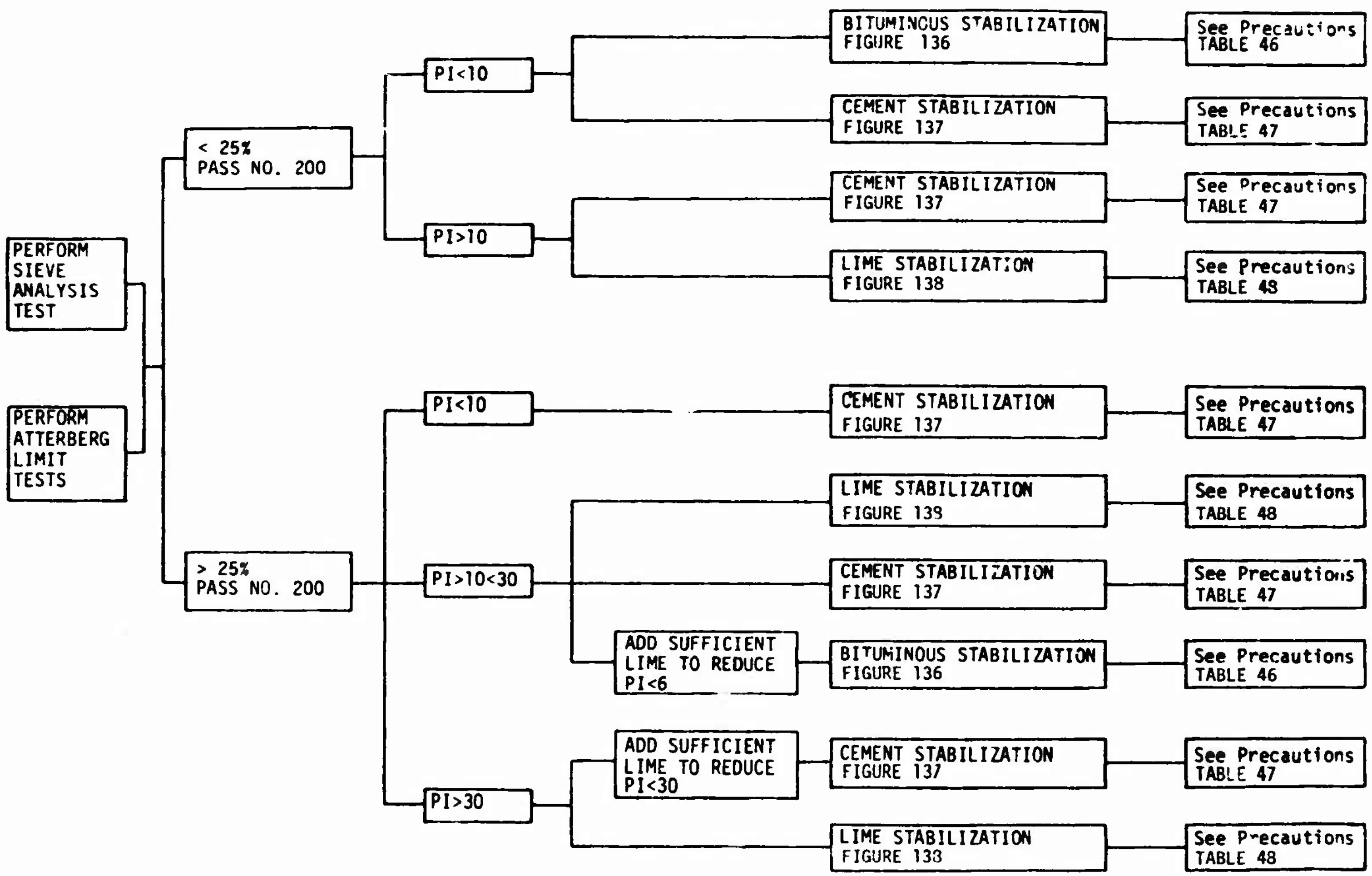


Figure 135 - Selection of Stabilizer for Nonexpedient Subgrade Construction

TABLE 46

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR BITUMINOUS STABILIZATION IN NONEXPEDIENT SUBGRADES

Condition	Precautions
Environmental	When cutbacks and emulsions are utilized, the air temperature and soil temperature should be above freezing. Bituminous materials should completely coat the soil particles before rainfall stops construction
Construction	Central batch plants, together with other specialized equipment, are necessary for bituminous stabilization with asphalt cements. Hot dry weather is preferred for all types of bituminous stabilization.

TABLE 47

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR CEMENT STABILIZATION IN NONEXPEDIENT SUBGRADES

Condition	Precautions
Environmental	If the soil temperature is less than 60 to 70°F and is not expected to increase for one month, chemical reactions will not occur rapidly, and strength gain of the cement-soil mixture will be minimal. If these environmental conditions are expected the cement may act as a soil modifier. Cement-soil mixtures should be scheduled for construction such that sufficient durability will be gained to resist any freeze-thaw cycles expected.
Construction	If heavy vehicles are allowed on the cement stabilized soils prior to a 10 to 14 day curing period, certain pavement damage can be expected.

TABLE 48

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR LIME STABILIZATION IN NONEXPEDIENT SUBGRADES

Condition	Precautions
Environmental	If the soil temperature is less than 60 to 70°F and is not expected to increase for one month, chemical reactions will not occur rapidly, and the strength gain of the lime-soil mixture will be minimal. If these environmental conditions are expected the lime may act as a soil modifier. Lime-soil mixtures should be scheduled for construction such that sufficient durability will be gained to resist any freeze-thaw cycles expected.
Construction	If heavy vehicles are allowed on the lime stabilized soils prior to a 10 to 14 day curing period, certain pavement damage can be expected.

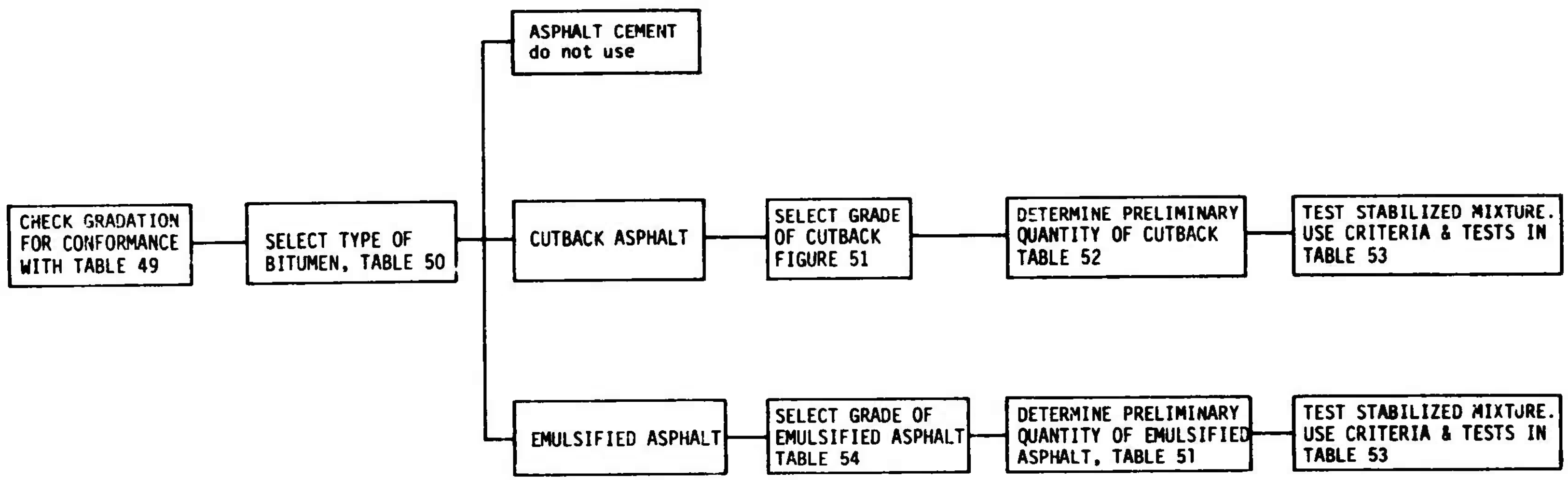


Figure 136 - Subsystem for Nonexpedient Subgrade Stabilization with Bituminous Materials

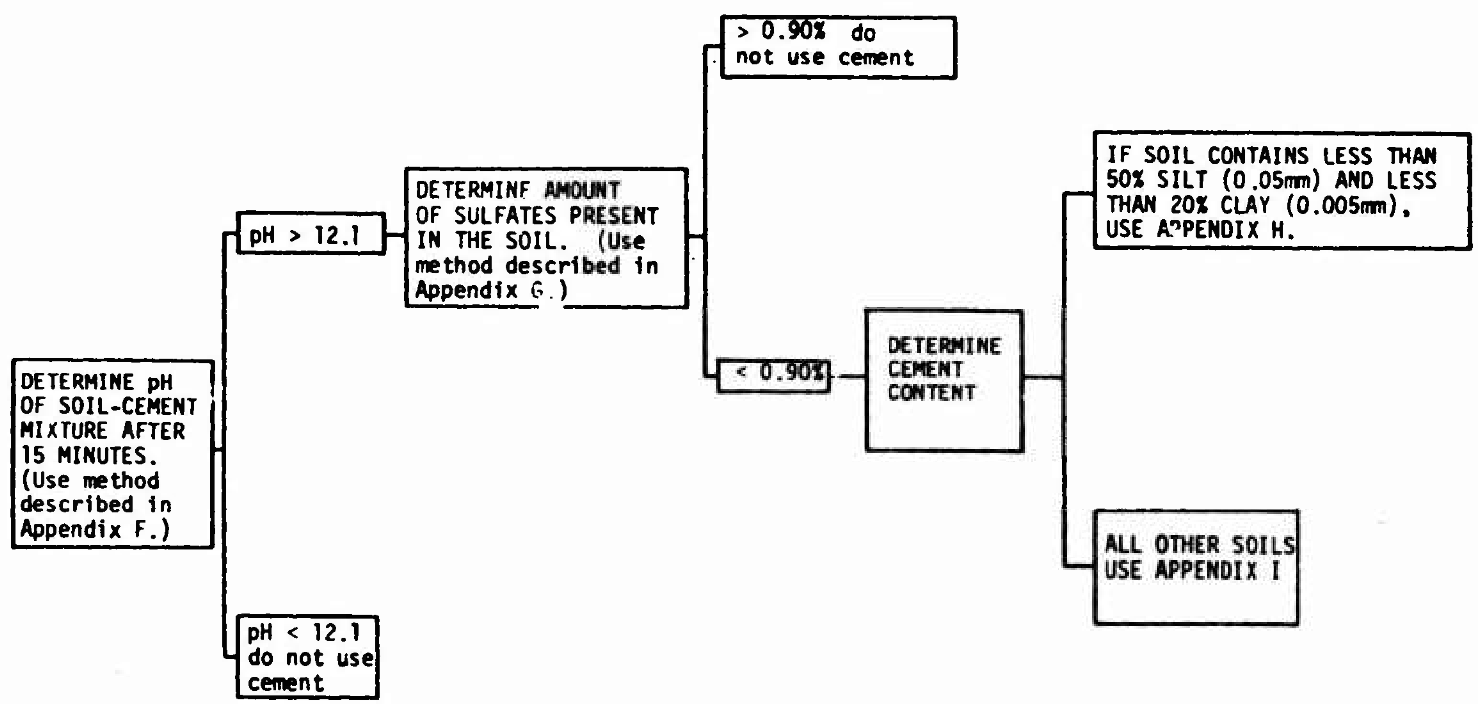


Figure 137 - Subsystem for Nonexpedient Subgrade Stabilization with Cement

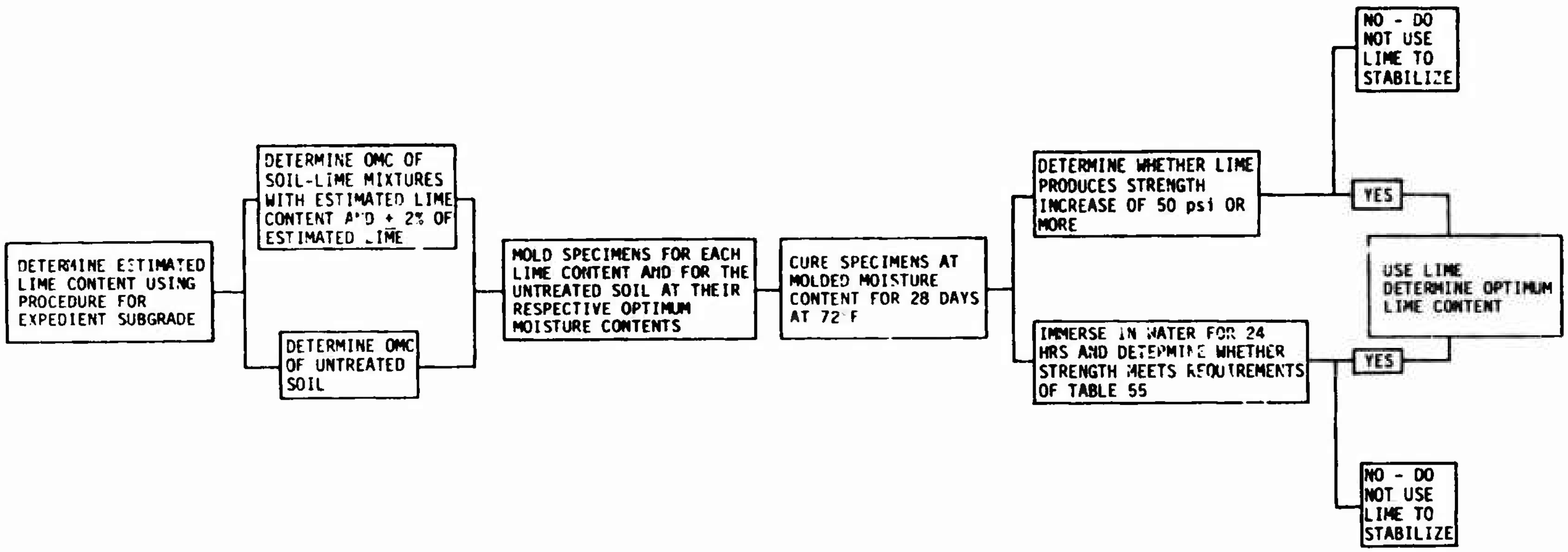
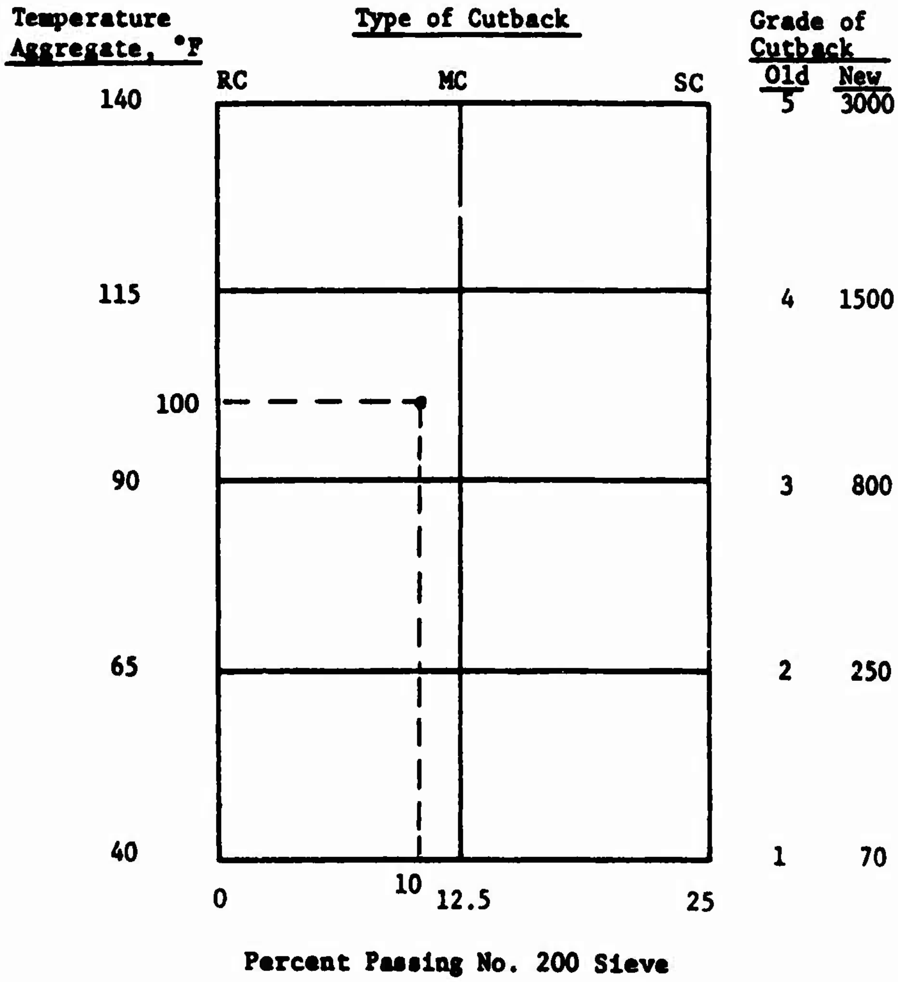


Figure 138 - Subsystem for Nonexpedient Subgrade Stabilization with Lime



Example: For aggregate temperature of 100°F and 10 percent passing No. 200 sieve, use MC 800 cutback.

Figure 139 - Selection of Type of Cutback for Stabilization

(from reference 38)

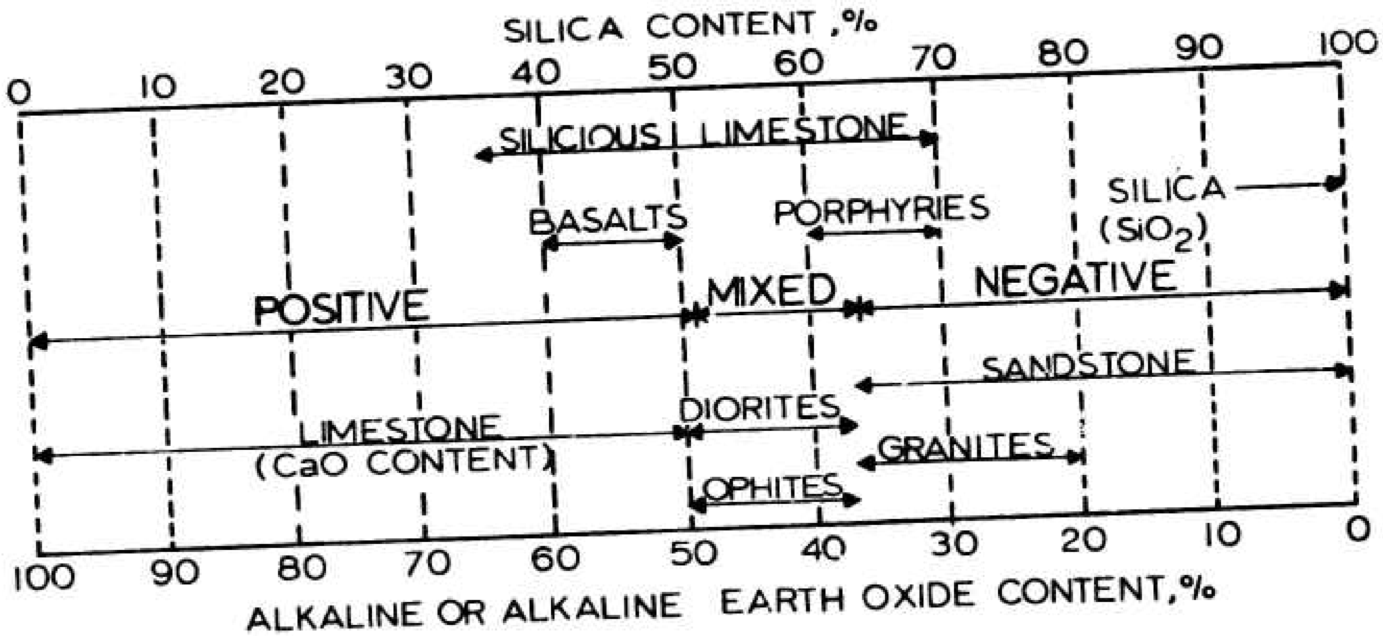


Figure 140 - Classification of Aggregates

(from reference 39)

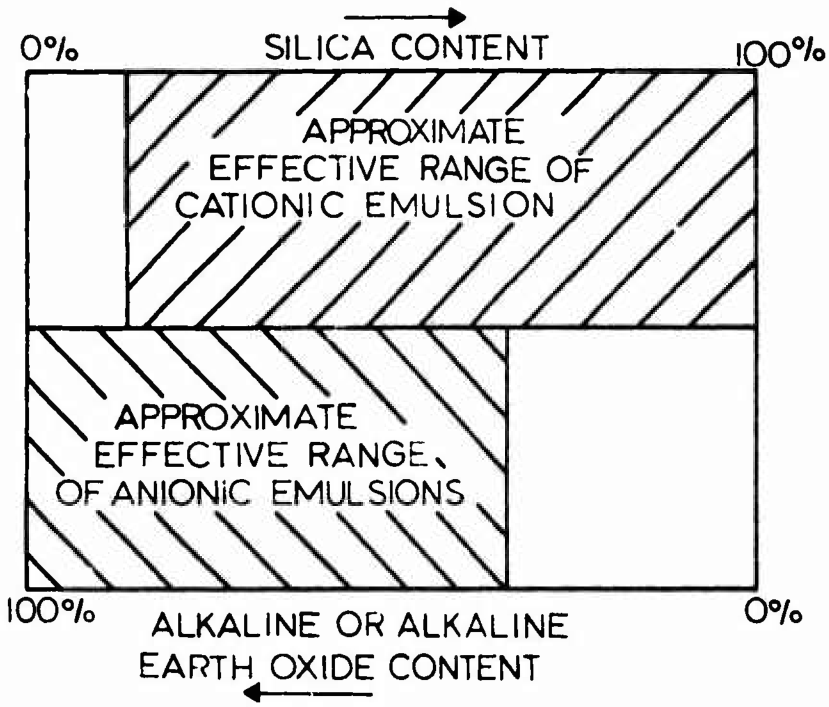


Figure 141 - Approximate Effective Range of Cationic and Anicnic Emulsions on Various Types of Aggregates

(from reference 39)

TABLE 49

ENGINEERING PROPERTIES OF MATERIALS

SUITABLE FOR BITUMINOUS STABILIZATION

Percent Passing Sieve	Sand-Bitumen	Soil-Bitumen	Sand-Gravel-Bitumen
1-1/2 inch			100
1 inch	100		
3/4 inch			60-100
No. 4	50-100	50-100	35-100
10	40-100		
40		35-100	13-50
100			8-35
200	5-12	good - 3-20 fair - 0-3 and 20-25	0-12

Includes slight modifications later made by Herrin.

(from reference 40)

TABLE 50

SELECTION OF A SUITABLE TYPE OF BITUMEN

FOR SOIL STABILIZATION PURPOSES

Sand Bitumen	Soil Bitumen	Crushed Stones and Sand-Gravel Bitumen
Cold Mix: Cutbacks See Figure 139 Emulsions See Table 54 See Figures 140 and 141 to determine if a catonic or anonic emulsion should be used	Cold Mix: Cutbacks See Figure 139 Emulsions See Table 54 See Figures 140 and 141 to determine if a catonic or anonic emulsion should be used	Cold Mix: Cutbacks See Figure 139 Emulsions See Table 54 See Figures 140 and 141 to determine if a catonic or anonic emulsion should be used

TABLE 51

EMULSIFIED ASPHALT REQUIREMENT

Percent passing No. 200	Lbs. of emulsified asphalt per 100 lbs. of dry aggregate when percent passing No. 10 sieve is:					
	50*	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

*50 or less.

(from reference 38)

TABLE 52

DETERMINATION OF QUANTITY OF CUTBACK ASPHALT

$p = 0.02\ (a) + 0.07\ (b) + 0.15\ (c) + 0.20\ (d)$	
Symbol	Definition
p	percent of residual asphalt by weight of dry aggregate
a	percent of mineral aggregate retained on No. 50 sieve
b	percent of mineral aggregate passing No. 50 and retained on No. 100 sieve
c	percent of mineral aggregate passing No. 100 and retained on No. 200 sieve
d	percent of mineral aggregate passing No. 200 sieve

TABLE 53

MARSHALL MIX DESIGN CRITERIA FOR

CUTBACK AND EMULSIFIED ASPHALT MIXTURES

Marshall Test	Criteria for a Test Temperature of 77°F	
	Minimum	Maximum
Stability, pounds	750	---
Flow, (0.01 inch)	7	16
Percent Air Voids	3	5

(from reference 42)

TABLE 54

SELECTION OF TYPE OF EMULSIFIED ASPHALT

FOR STABILIZATION

Percent Passing No. 200 Sieve	Relative Water Content of Soil	
	Wet (5 percent +)	Dry (0-5 percent)
0-5	SS-1h (CSS-1h)	CMS-2h (or SS-1h*)
5-15	SS-1, SS-1h (CSS-1, CSS-1h)	CMS-2h (or SS-1h*, SS-1*)
15-25	SS-1 (CSS-1)	CMS-2h

*Soil should be pre-wetted with water before using these types of emulsified asphalts.

(from reference 38)

TABLE 55

TENTATIVE SHORT-TERM IMMERSED STRENGTH REQUIREMENTS

FOR SOIL-LIME MIXTURES

Anticipated Use	Residual Strength Requirement, psi[a]	Short-Term Immersed Strength[b] Requirements
Modified Subgrade	20	30
Subbase		
Rigid Pavement	20	30
Flexible Pavement		
Thickness of Cover		
10 inches	30	45
8 inches	40	55
5 inches	60	80
Base	100	130

[a]As recommended by Thompson (reference 10).

[b]Unconfined compressive strength of 2- by 4-inch specimen after short-term immersion.

APPENDIX D

NONEXPEDIENT BASE COURSE STABILIZATION SYSTEM

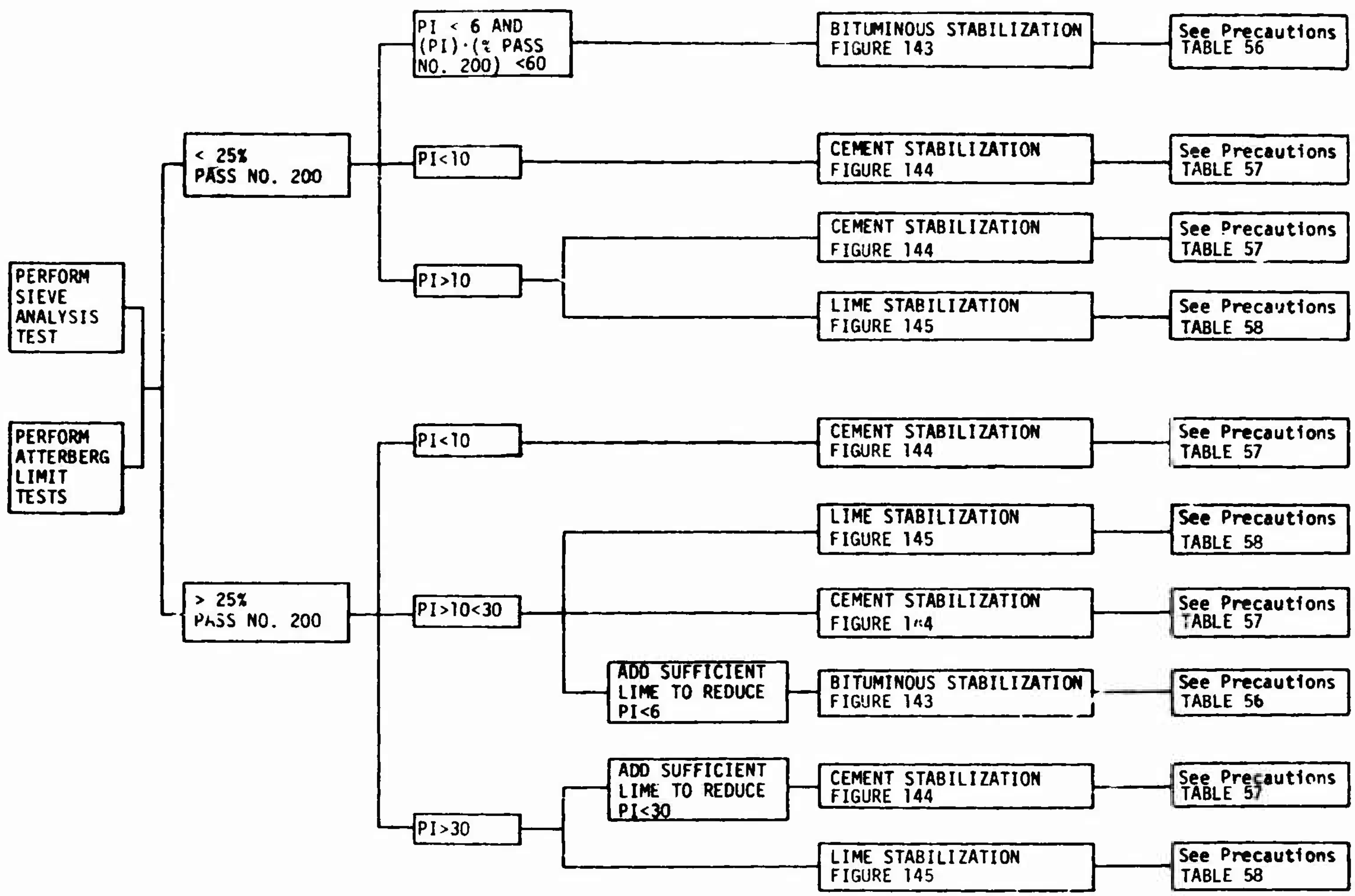


Figure 142 - Selection of Stabilizer for Nonexpedient Base Construction

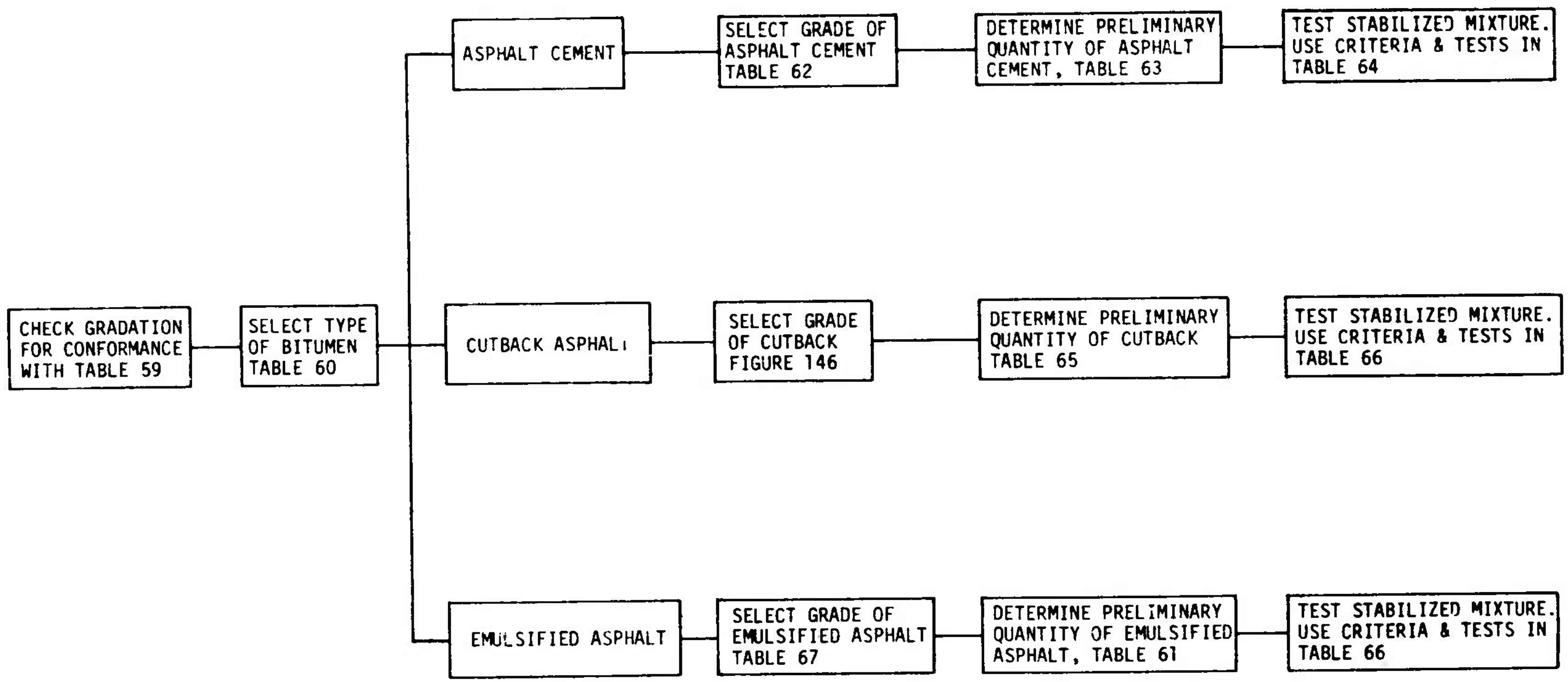


Figure 143 - Subsystem for Nonexpedient Base Course Stabilization with Bituminous Materials

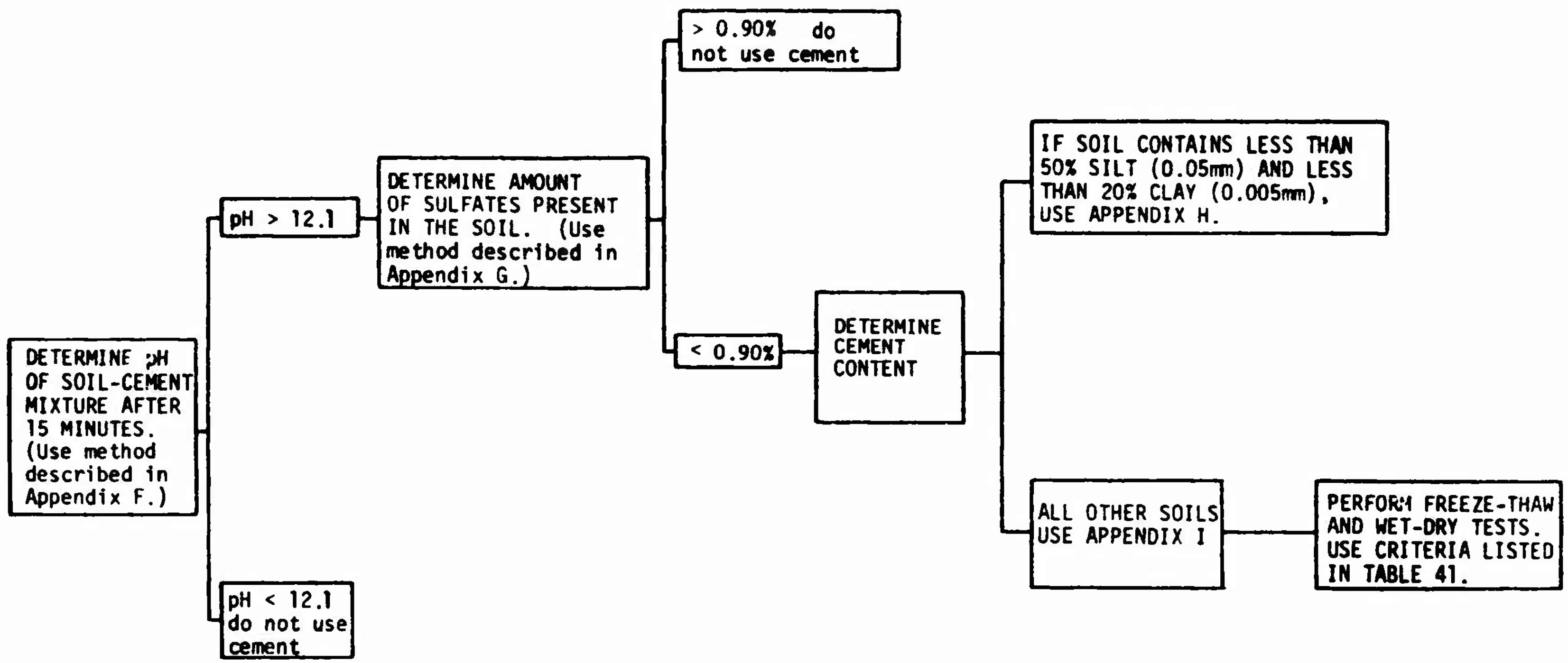


Figure 144 - Subsystem for Nonexpedient Base Course Stabilization with Cement

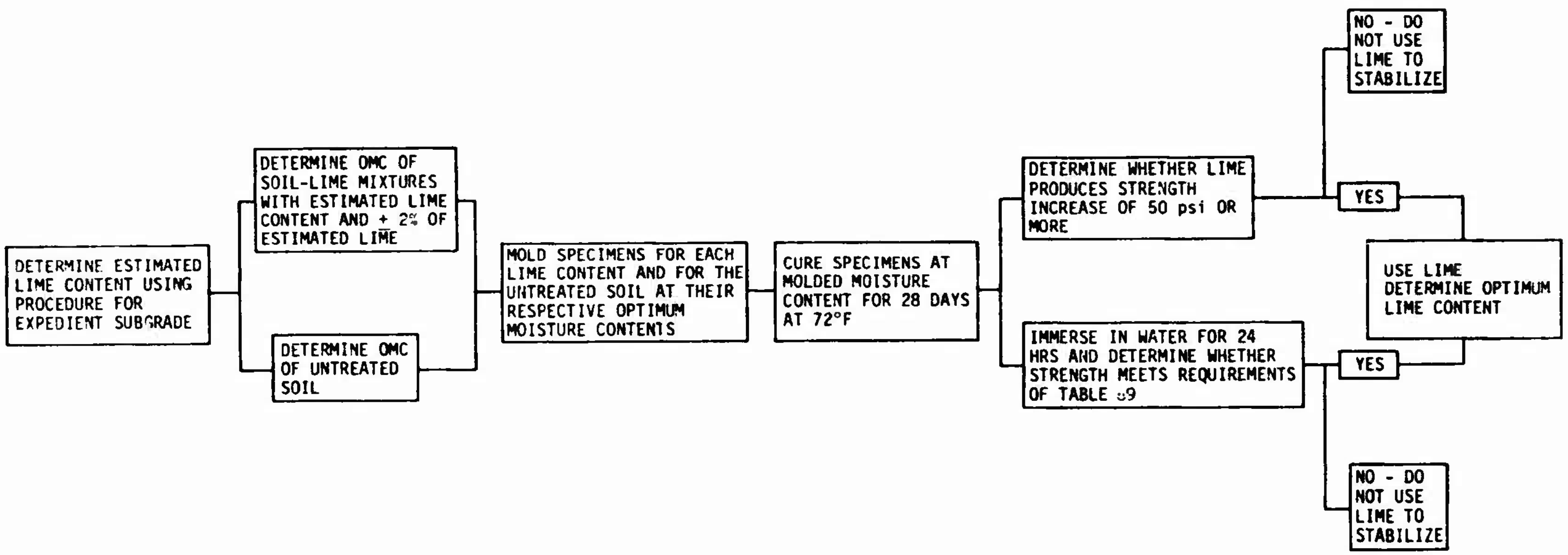


Figure 145 - Subsystem for Nonexpedient Base Course Stabilization with Lime

TABLE 56

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR BITUMINOUS STABILIZATION IN NONEXPEDIENT BASE COURSES

Condition	Precautions
Environmental	When asphalt cements are used for bituminous stabilization, proper compaction must be obtained. If thin lifts of asphalt concrete are being placed, the air temperature should be 40°F and rising, and compaction equipment should be used immediately after lay down operation. Adequate compaction can be obtained at freezing temperatures if thick lifts are utilized. When cutbacks and emulsions are utilized, the air temperature and soil temperature should be above freezing. Bituminous materials should completely coat the soil particles before rainfall stops construction.
Construction	Central batch plants, together with other specialized equipment, are necessary for bituminous stabilization with asphalt cements. Hot dry weather is preferred for all types of bituminous stabilization.

TABLE 57

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR CEMENT STABILIZATION IN NONEXPEDIENT BASE COURSES

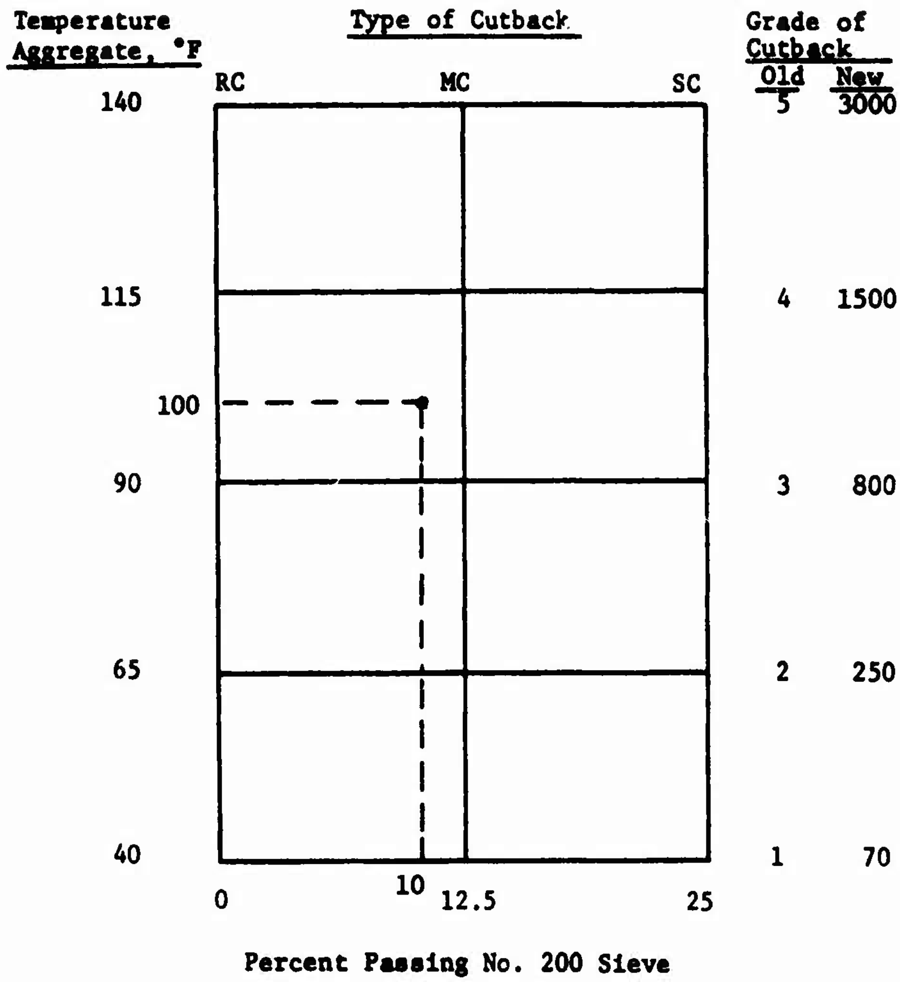
Condition	Precautions
Environmental	If the soil temperature is less than 60° to 70°F and is not expected to increase for one month, chemical reactions will not occur rapidly, and strength gain of the cement-soil mixture will be minimal. If these environmental conditions are expected the cement may be expected to act as a soil modifier. Cement-soil mixtures should be scheduled for construction such that sufficient durability will be gained to resist any freeze-thaw cycles expected.
Construction	If heavy vehicles are allowed on the cement stabilized soils prior to a 10 to 14 day curing period, certain pavement damage can be expected.

TABLE 58

ENVIRONMENTAL AND CONSTRUCTION PRECAUTIONS

FOR LIME STABILIZATION IN NONEXPEDIENT BASE COURSES

Condition	Precautions
Environmental	If the soil temperature is less than 60° to 70°F and is not expected to increase for one month, chemical reactions will not occur rapidly, and the strength gain of the lime-soil mixture will be minimal. If these environmental conditions are expected the lime may be expected to act as a soil modifier. Lime-soil mixtures should be scheduled for construction such that sufficient durability will be gained to resist any freeze-thaw cycles expected.
Construction	If heavy vehicles are allowed on the lime stabilized soils prior to 10 to 14 day curing period, certain pavement damage can be expected.



Example: For aggregate temperature of 100°F and 10 percent passing No. 200 sieve, use MC 800 cutback.

Figure 146 - Selection of Type of Cutback for Stabilization

(from reference 38)

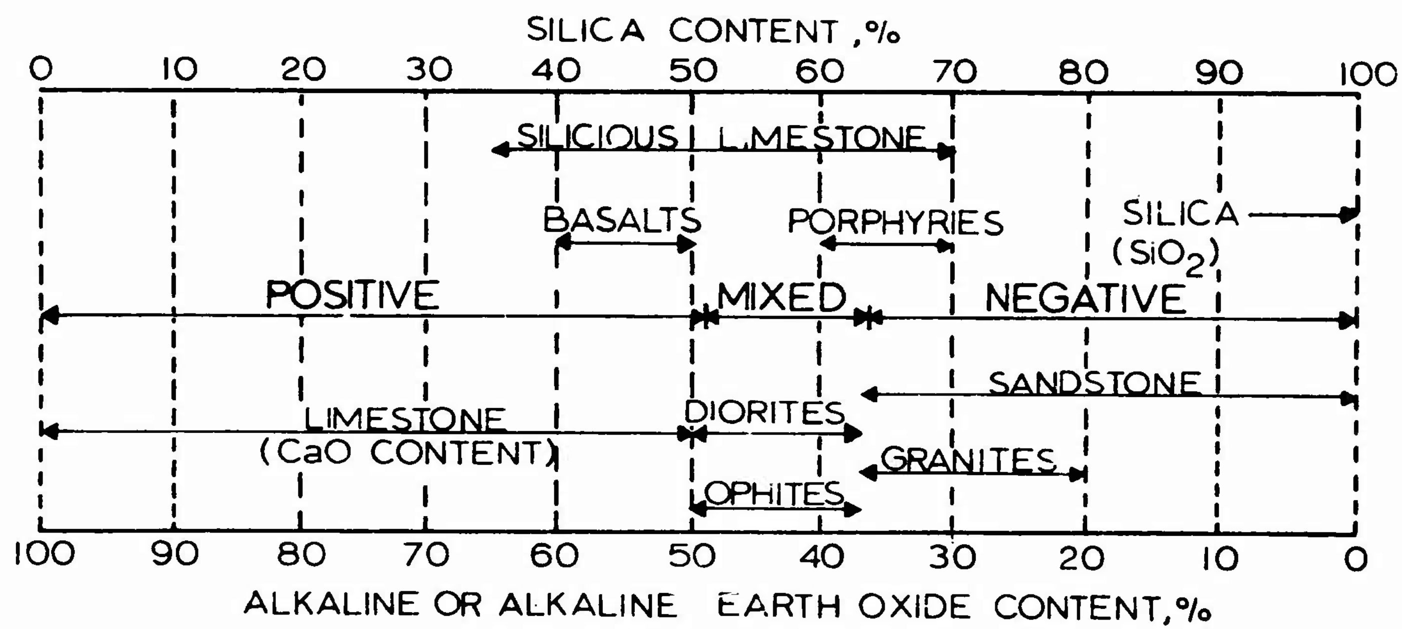


Figure 147 - Classification of Aggregates

(from reference 39)

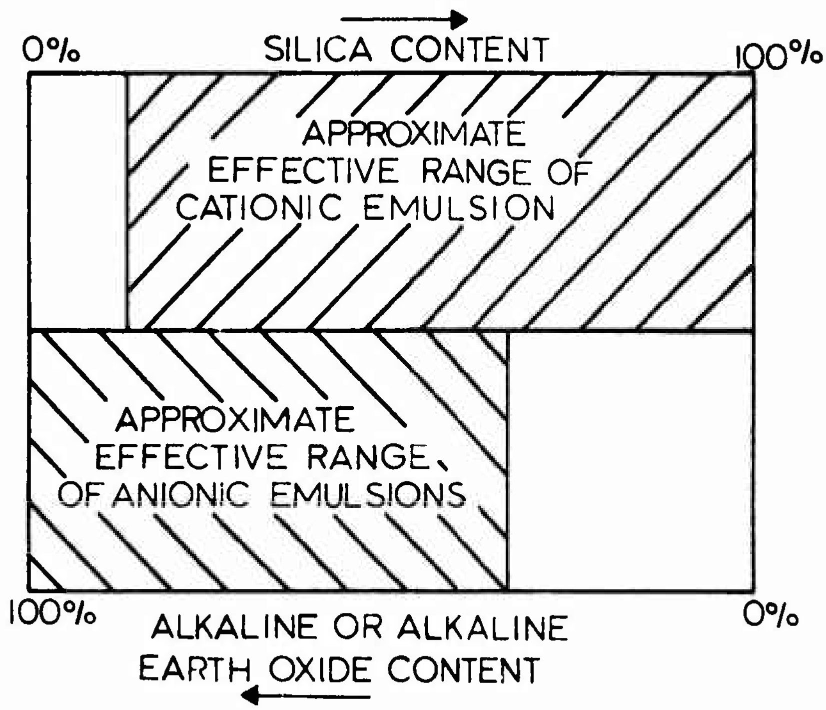


Figure 148 - Approximate Effective Range of Cationic and Anionic Emulsions on Various Types of Aggregates

(from reference 39)

TABLE 59

ENGINEERING PROPERTIES OF MATERIALS

SUITABLE FOR BITUMINOUS STABILIZATION

Percent Passing Sieve	Sand-Bitumen	Soil-Bitumen	Sand-Gravel-Bitumen
1-1/2 inch			100
1 inch	100		
3/4 inch			60-100
No. 4	50-100	50-100	35-100
10	40-100		
40		35-100	13-50
100			8-35
200	5-12	good - 3-20 fair - 0-3 and 20-25	0-12

Includes slight modifications later made by Herrin.

(from reference 40)

TABLE 60

SELECTION OF A SUITABLE TYPE OF BITUMEN FOR SOIL STABILIZATION PURPOSES

Sand Bitumen	Soil Bitumen	Crushed Stones and Sand-Gravel Bitumen
Hot Mix: Asphalt Cements 60-70 hot climate 85-100 120-150 cold climate		Hot Mix: Asphalt Cements 40-50 hot climate 60-70 85-100 cold climate
Cold Mix: Cutbacks See figure 146	Cold Mix: Cutbacks See figure 146	Cold Mix: Cutbacks See figure 146
Emulsions See table 67 See figures 147 and 148 to determine if a cationic or anionic emulsion should be used	Emulsions See table 67 See figures 147 and 148 to determine if a cationic or anionic emulsion should be used	Emulsions See table 67 See figures 147 and 148 to determine if a cationic or anionic emulsion should be used

TABLE 61

EMULSIFIED ASPHALT REQUIREMENT

Percent passing No. 200	Lbs. of emulsified asphalt per 100 lbs. of dry aggregate when percent passing No. 10 sieve is:					
	50*	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

*50 or less.

(from reference 38)

TABLE 62

DETERMINATION OF ASPHALT GRADE FOR

BASE COURSE STABILIZATION

Pavement Temperature Index*	Asphalt Grade, Penetration
Negative	100-120
0-40	85-100
40-100	60-70
Above 100	40-50

*The sum, for a 1-year period, of the increments above 75°F of monthly averages of the daily maximum temperatures. Average daily maximum temperatures for the period of record should be used where 10 or more years of record are available. For records of less than 10-year duration the record for the hottest year should be used. A negative index results when no monthly average exceeds 75°F. Negative indexes are evaluated merely be subtracting the largest monthly average from 75°F.

TABLE 63

SELECTION OF ASPHALT CEMENT CONTENT

FOR NONEXPEDIENT BASE COURSE CONSTRUCTION

Aggregate Shape and Surface Texture	Percent Asphalt by Weight of Dry Aggregate*
Rounded and Smooth	4
Angular and Rough	6
Intermediate	5

*Approximate quantities which may be adjusted in field based on observation of mix and engineering judgment.

TABLE 64

SUGGESTED MARSHALL MIX DESIGN CRITERIA*

TEST PROPERTY	TIRE PRESSURE	
	100 PSI	200 PSI
Stability, pounds	500 or more	1800 or more
Flow, 0.01 inch	20 or less	16 or less
Air Voids	4-6	5-7
Percent Voids Filled with Asphalt	65-75	70-80

*Tests for compliance to criteria should be conducted at a temperature representative of a critical in-service condition.

TABLE [illegible]

DETERMINATION OF QUANTITY OF CUTBACK ASPHALT

$p = 0.02\,(a) + 0.07\,(b) + 0.15\,(c) + 0.20\,(d)$	
Symbol	Definition
p	percent of residual asphalt by weight of dry aggregate*
a	percent of mineral aggregate retained on No. 50 sieve
b	percent of mineral aggregate passing No. 50 and retained on No. 100 sieve
c	percent of mineral aggregate passing No. 100 and retained on No. 200 sieve
d	percent of mineral aggregate passing No. 200 sieve

*Percent cutback can be obtained by referring to Table 68 and utilizing the following equation:

$$\text{percent cutback} = \frac{\text{percent residual asphalt (p)}}{(100 - \text{percent solvent})} \times 100$$

TABLE 66

MARSHALL MIX DESIGN CRITERIA FOR

CUTBACK AND EMULSIFIED ASPHALT MIXTURES

Marshall Test	Criteria for a Test Temperature of 77°F	
	Minimum	Maximum
Stability, pounds	750	---
Flow, (0.01 inch)	7	16
Percent Air Voids	3	5

TABLE 67

SELECTION OF TYPE OF EMULSIFIED ASPHALT

FOR STABILIZATION

Percent Passing No. 200 Sieve	Relative Water Content of Soil	
	Wet (5 percent +)	Dry (0-5 percent)
0-5	SS-1h (CSS-1h)	CMS-2h (or SS-1h*)
5-15	SS-1, SS-1h (CSS-1, CSS-1h)	CMS-2h (or SS-1h*, SS-1*)
15-25	SS-1 (CSS-1)	CMS-2h

*Soil should be pre-wetted with water before using these types of emulsified asphalts.

(from reference 38)

TABLE 68

ASPHALT CUTBACK COMPOSITION

Type of Cutback	Solvent	Percent Solvent for Particular Grades				
		30	70	250	800	3000
RC	Gasoline or Naptha	--	35	25	17	13
MC	Kerosene	46	36	26	19	14
SC	Fuel Oil	--	50	40	30	20

TABLE 69

PORTLAND CEMENT ASSOCIATION CRITERIA FOR

SOIL-CEMENT MIXTURES USED IN BASE COURSE

Soil Classification		Soil-Cement Weight Loss During 12 Cycles of either Wet-Dry Test or Freeze-Thaw Test
AASHO	Unified*	
A-1	GW, GP, GM SW, SP, SM	less than or equal to 14 percent
A-2-4, A-2-5	GM, GC, SM, SC	
A-3	SP	
A-2-6, A-2-7	GM, GC, SM, SC	less than or equal to 10 percent
A-4	CL, ML	
A-5	ML, MH, OH	
A-6	CL, CH	less than or equal to 7 percent
A-7	OH, MH, CH	

*based on correlation presented by Air Force

(from reference 24)

TABLE 70

TENTATIVE SHORT-TERM IMMERSED STRENGTH REQUIREMENTS

FOR SOIL-LIME MIXTURES

Anticipated Use	Residual Strength Requirement, psi[a]	Short-Term Immersed Strength[b] Requirements
Modified Subgrade	20	30
Subbase		
Rigid Pavement	20	30
Flexible Pavement		
Thickness of Cover		
10 inches	30	45
8 inches	40	55
5 inches	60	80
Base	100	130

[a]As recommended by Thompson (reference 10).

[b]Unconfined compressive strength of 2- by 4-inch specimen after short-term immersion.

APPENDIX E

RAPID TEST PROCEDURES FOR EXPEDIENT CONSTRUCTION OPERATIONS USING SOIL-CEMENT STABILIZATION*

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A RAPID method of testing soil-cement has been used successfully for emergency construction and for very small projects where more complete testing is not feasible or practical. The engineer applying this procedure should be familiar with the details of the ASTM-AASHO soil-cement test methods described in Chapter 3 so that he can properly interpret and evaluate the data obtained with this rapid method.

The following steps, which are described in more detail in the following paragraphs, are suggested:

1. Determine the maximum density and optimum moisture content for the soil-cement mixture.
2. Mold specimens for inspection of hardness.
3. Inspect specimens using "pick" and "click" procedures.

Moisture-Density Test

The maximum density and optimum moisture content are determined at 12 per cent cement by weight by means of the modified moisture-density test procedure described in Chapter 3.

In instances where the standard mold and rammer are not available, tests can be made by using a 2-in. diameter filled-in gas pipe of sufficient length to weigh 5.5 lb. as the compacting rammer and a No. 2½ tin can as the mold.

With experience the optimum moisture can be determined quite closely by "feel." When squeezed, soil-cement at optimum moisture will form a cast that will stick together when it is handled.

Molding Specimens

Specimens for inspection of hardness are molded by the same procedure described in Chapter 3. These specimens generally contain 10, 14 and 18 per cent cement by weight. It is best if these specimens can be molded in the standard mold, and then removed from the mold and placed in high humidity for hydration.

However, if a standard mold is not available it is possible to mold these specimens in No. 2½ tin cans, using the compacting rammer suggested above. The tin-can mold can be torn or ripped from the hardened soil-cement specimens with pliers after a few days.

Inspecting Specimens

After at least a day or two of hardening, during which they are kept moist, and after a 3-hour soaking, the specimens are inspected by "picking" with a sharp-pointed instrument and by sharply "clicking" each specimen against a hard object such as concrete to determine their relative hardness when wet.

"Pick" Test

In the pick test, the specimen is held in one hand and a relatively sharp-pointed instrument, such as a dull ice pick, is lightly jabbed into the specimen (or the end of a specimen molded in a can) from a distance of two or three inches. If the specimen resists this light picking, the force of impact is increased until the pick is striking the specimen with considerable force. Specimens that are hardening satisfactorily will definitely resist the penetration of the pick,

The "pick" test.

*Since this material has been taken directly from the Portland Cement Association text, figure numbers and certain other references in this Appendix will not be in agreement with other portions of this report.

whereas specimens that are not hardening properly will resist little. To pass the pick test, a specimen that is not over 7 days old and that has been soaked in water must prevent the penetration of the ice pick, which is under considerable force, to a distance greater than about one-eighth to one-quarter inch.

"Click" Test

The click test is then applied to water-soaked specimens that are apparently hardening satisfactorily and that have passed the pick test. In the click test, the specimens are held perpendicular to each other and about four inches apart, one in each hand. They are then lightly clicked together a number of times, the force of impact being increased with each click. Specimens that are hardening satisfactorily will click together with a "ringing" or "solid" tone. As the force of impact is increased, one of the specimens may break transversely even though it is hardening adequately. The internal portion of a satisfactory specimen should then pass the pick test. When two or three hard specimens are once obtained they may be saved and one may be used in the click test with a soil-cement specimen of a soil in the process of being tested.

When a poorly hardened specimen is clicked with a satisfactory specimen, a "dull thud" sound is obtained rather than the "solid" sound obtained with two satisfactory specimens. After the first or second click the inferior specimen will generally break and its internal portion will not pass the pick test.

The "click" test.

At the time the click test is made, the age of the specimens must be taken into account. For instance, specimens that are not properly hardened at an age of 4 days may be satisfactorily hardened at an age of 7 days.

The above pick and click procedures are then repeated after the specimens have been dried out and again after a second soaking in order to test their relative hardness at both extremes of moisture content.

If equipment is available for making compression tests, these tests will provide further valuable data for study. It is suggested that duplicate specimens be molded and tested in compression at the age of 7 days and after a soaking in water for 4 hours. A satisfactory soil-cement mixture will have a compressive strength of about 400 lb. per sq.in. or more.

General Remarks

There is a distinct difference between satisfactorily hardened soil-cement specimens and inadequately hardened specimens. Even an inexperienced tester will soon be able to differentiate between them and to select a safe cement content to harden the soil. It is important to remember that an excess of cement is not harmful but that a deficiency of cement will result in inferior soil-cement.

If the 10 and 14 per cent specimens are apparently hardening satisfactorily and compression-test data are favorable, the project can immediately be started using a cement content of 12 per cent by weight. If the quantities of cement available for construction are limited and if the 10 per cent cement specimens are hard and have good compressive strength, additional specimens should be molded at 8 per cent cement, be permitted to hydrate and then be tested in the same manner as the other specimens. If the 8 per cent cement specimens are satisfactorily hardened, the cement content being used in construction can be reduced to 10 per cent.

Should a 10 per cent specimen be comparatively soft at 4 days' hydration, while the 14 and 18 per cent specimens are hardening satisfactorily, construction should be started using 16 per cent cement by weight until additional data are obtained.

In some unusual instances, the 18 per cent cement specimen may not harden satisfactorily. The engineer then has two alternatives: (1) the effect of higher cement contents may be investigated to see whether 22 or 26 per cent cement will harden the soil; or (2) a borrow soil requiring a relatively low cement factor may be located and hauled to the runway or roadway to "cap" the poor soil. The latter procedure will generally be the more economical one.

APPENDIX F

pH TEST ON SOIL-CEMENT MIXTURES

Materials:

1. Portland cement to be used for soil stabilization

Apparatus:

1. pH meter (the pH meter must be equipped with an electrode having a pH range of 14)
2. 150 ml. plastic bottles with screw-top lids
3. 50 ml. plastic beakers
4. Distilled water
5. Balance
6. Oven
7. Moisture cans

Procedure:

1. Standardize the pH meter with a buffer solution having a pH of 12.00.
2. Weigh to the nearest 0.01 gms., representative samples of air-dried soil, passing the No. 40 sieve and equal to 25.0 gms. of oven-dried soil.
3. Pour the soil samples into 150 ml. plastic bottles with screw-top lids.
4. Add 2.5 gms. of the portland cement.
5. Thoroughly mix soil and portland cement.
6. Add sufficient distilled water to make a thick paste. (Caution: too much water will reduce the pH and produce an incorrect result.)
7. Stir the soil-cement and water until thorough blending is achieved.

8. After 15 minutes, transfer part of the paste to a plastic beaker and measure the pH.

9. If the pH is 12.1 or greater, the soil organic matter content should not interfere with the cement stabilizing mechanism. To determine the required percent of cement, refer to design methods outlined in Figures 123, 130, 137 or 144 as appropriate.

APPENDIX G

DETERMINATION OF SULFATE IN SOILS

GRAVIMETRIC METHOD

Scope

Applicable to all soil types with the possible exception of soils containing certain organic compounds. This method should permit the detection of as little as 0.05 percent sulfate as SO_4.

Reagents

1. Barium chloride, 10 percent solution of $BaCl_2 \cdot 2H_2O$. (Add 1 ml. 2 percent HCl to each 100 ml. of solution to prevent formation of carbonate.)
2. Hydrochloric acid, 2 percent solution (0.55 N)
3. Magnesium chloride, 10 percent solution of $MgCl_2 \cdot 6H_2O$
4. Demineralized water
5. Silver nitrate, 0.1 N solution

Apparatus

1. Beaker, 1000 ml.
2. Burner and ring stand
3. Filtering flask, 500 ml.
4. Buchner funnel, 9 cm.
5. Filter paper, Whatman No. 40, 9 cm.
6. Filter paper, Whatman No. 42, 9 cm.
7. Saranwrap
8. Crucible, ignition, or aluminum foil, heavy grade
9. Analytical balance
10. Aspirator or other vacuum source

Procedure

1. Select a representative sample of air-dried soil weighing approximately 10 gm. Weigh to the nearest 0.01 gm. (Note: When sulfate content is anticipated to be less than 0.1 percent, a sample weighing 20 gm. or more may be used.) (The moisture content of the air-dried soil must be known for later determination of dry weight of the soil.)
2. Boil for 1-1/2 hours in beaker with mixture of 300 ml. water and 15 ml. HCl.
3. Filter through Whatman No. 40 paper, wash with hot water, dilute combined filtrate and washings to 50 ml.
4. Take 100 ml. of this solution and add $MgCl_2$ solution until no more precipitate is formed.
5. Filter through Whatman No. 42 paper, wash with hot water, dilute combined filtrate and washings to 200 ml.
6. Heat 100 ml. of this solution to boiling and add $BaCl_2$ solution very slowly until no more precipitate is formed. Continue boiling for about 5 minutes and let stand overnight in warm place, covering beaker with Saranwrap.
7. Filter through Whatman No. 42 paper. Wash with hot water until free from chlorides (filtrate should show no precipitate when a drop of $AgNO_3$ solution is added).
8. Dry filter paper in crucible or on sheet of aluminum foil. Ignite paper. Weigh residue on analytical balance as $BaSO_4$.

Calculation

$$\text{Percent } SO_4 = \frac{\text{Weight of residue}}{\text{Oven-dry weight of initial sample}} \times 411.6$$

where

$$\text{Oven-dry weight of initial sample} = \frac{\text{Air-dry weight of initial sample}}{1 + \frac{\text{Air-dry moisture content (percent)}}{\text{100 Percent}}}$$

Note

If precipitated from cold solution, barium sulfate is so finely dispersed that it can not be retained when filtering by the above method. Precipitation from a warm, dilute solution will increase crystal size. Due to the absorption (occlusion) of soluble salts during the precipitation of $BaSO_4$ a small error is introduced. This error can be minimized by permitting the precipitate to digest in a warm, dilute solution for a number of hours. This allows the more soluble small crystals of $BaSO_4$ to dissolve and recrystallize on the larger crystals.

DETERMINATION OF SULFATE IN SOILS

TURBIDIMETRIC METHOD

Reagents:

1. Barium chloride crystals (Grind analytical reagent grade barium chloride to pass a 1 mm. sieve.)
2. Ammonium acetate solution (0.5N) (Add dilute hydrochloric acid until the solution has a pH of 4.2.)
3. Distilled water

Apparatus:

1. Moisture can
2. Oven
3. 200 ml. beaker
4. Burner and ring stand
5. Filtering flask
6. Buchner funnel, 9 cm.
7. Filter paper, Whatman No. 40, 9 cm.
8. Vacuum source
9. Spectrophotometer and standard tubes (Bausch and Lombe Spectronic 20 or equivalent)
10. pH meter

Procedure:

1. Take a representative sample of air-dried soil weighing approximately 10 gms., and weigh to the nearest 0.01 gms. (The moisture content of the air-dried soil must be known for later determination of dry weight of the soil.)
2. Add the ammonium acetate solution to the soil. (The ratio of soil to solution should be approximately 1:5 by weight.)
3. Boil for about 5 minutes.
4. Filter through Whatman No. 40 filter paper. If the extracting solution is not clear, filter again.
5. Take 10 ml. of extracting solution (this may vary depending on the concentration of sulfate in the solution) and dilute with distilled water to about 40 ml. Add about 0.2 gm. of barium chloride crystals and dilute to make the volume exactly equal to 50 ml. Stir for 1 minute.
6. Immediately after the stirring period has ended, pour a portion of the solution into the standard tube and insert the tube into the cell of the spectrophotometer. Measure the turbidity at 30 sec. intervals for 4 minutes. Maximum turbidity is usually obtained within 2 minutes and the readings remain constant thereafter for 3-10 minutes. Consider the turbidity to be the maximum reading obtained in the 4 minute interval.
7. Compare the turbidity reading with a standard curve and compute the sulfate concentration (as SO_4) in the original extracting solution. (The standard curve is secured by carrying out the procedure with standard potassium sulfate solutions.)
8. Correction should be made for the apparent turbidity of the samples by running blanks in which no barium chloride is added.

Sample Calculation:

Given: Weight of air-dried sample = 10.12 gms.

Water Content = 9.36 percent

Weight of dry soil = 9.27 gms.

Total volume of extracting solution = 39.1 ml.

10 ml. of extracting solution was diluted to 50 ml. after addition of barium chloride (see step 5). The solution gave a transmission reading of 81.

Calculation:

From the standard curve, a transmission reading of 81 corresponds to 16.0 ppm. (see following figure).

$\therefore$ Concentration of original extracting solution = 16.0 x 5 = 80.0 ppm.

$$\text{Percent } SO_4^{--} = \frac{80.0 \times 39.1 \times 100}{1000 \times 1000 \times 9.27} = 0.0338 \text{ percent}$$

Determination of Standard Curve:

1. Prepare sulfate solutions of 0, 4, 8, 12, 16, 20, 25, 30, 35, 40, 45, 50 ppm. in separate test tubes. The sulfate solution is made from potassium sulfate salt dissolved in 0.5 N ammonium acetate (with pH adjusted to 4.2).
2. Continue Steps 5 and 6 in the procedure as described in Determination of Sulfate in Soil by Turbidimetric Method.
3. Draw standard curve as shown in following figure by plotting transmission readings for known concentrations of sulfate solutions.

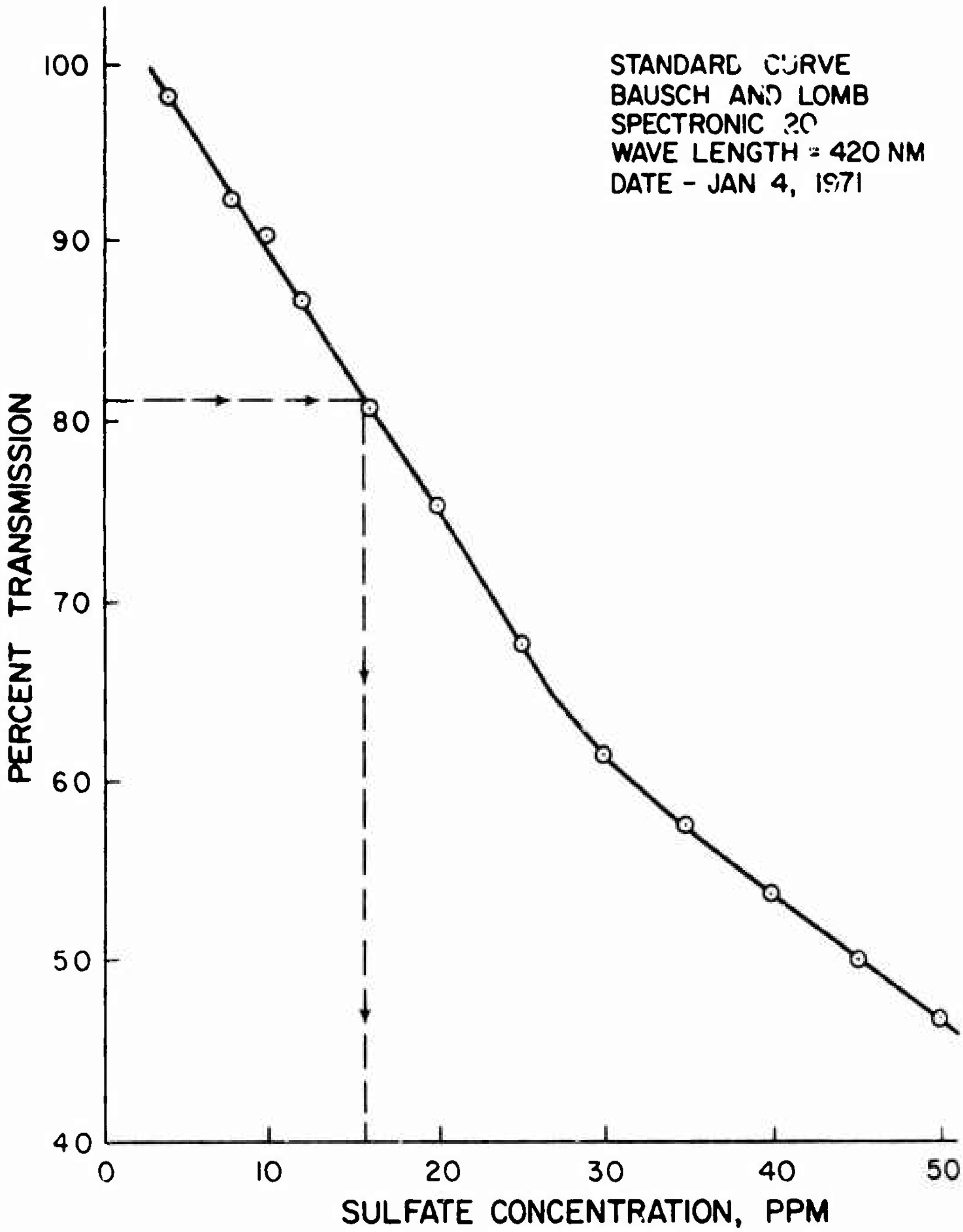


Figure 149 - Example Standard Curve for Spectrophotometer

APPENDIX H

SELECTION OF CEMENT CONTENT FOR CEMENT STABILIZED SANDY SOIL*

Reproduced with permission of the Portland Cement Association (reference 24)

THE following short-cut test procedures for sandy soils were developed as a result of a correlation made by the Portland Cement Association of the data obtained from ASTM-AASHO tests on 2,438 sandy soils.* These procedures do not involve new tests or additional equipment. Instead, some tests can be eliminated by the use of charts developed in previous tests on similar soils. The only tests required are a grain-size analysis, a moisture-density test and compressive-strength tests. Relatively small samples are needed. All tests, except for the 7-day compressive-strength tests, can be completed in one day.

Two procedures are used: Method A for soils not containing material retained on the No. 4 sieve and Method B for soils containing material retained on the No. 4 sieve. Method B was recently developed to permit the use of moisture-density data obtained on the total soil-cement mixture, as specified by the ASTM-AASHO moisture-density test methods revised in 1957.

The procedures can be used only with soils containing less than 50 per cent material smaller than 0.05 mm. (silt and clay) and less than 20 per cent material smaller than 0.005 mm. (clay). These were the gradation limits for the soils that were included in the correlation used to develop the original charts. Dark grey to black soils with appreciable amounts of organic impurities were not included in the correlation and therefore cannot be tested by these procedures. This is also true of miscellaneous granular materials such as cinders, caliche, chat, chert, marl, red dog, scoria, shale, slag, etc. Moreover, the short-cut procedures cannot be used with granular soils containing material retained on the No. 4 sieve if that material has a bulk specific gravity less than 2.45.

The short-cut test procedures do not always indicate the minimum cement factor that can be used with a particular sandy soil. However, they almost always provide a safe cement factor, generally close to that indicated by standard ASTM-AASHO wet-dry and freeze-thaw tests.

The procedures are being widely applied by engineers and builders and may largely replace the standard tests when experience in their use is gained and the relationships are checked. The charts and procedures may be modified to conform to local climatic and soil conditions if necessary.

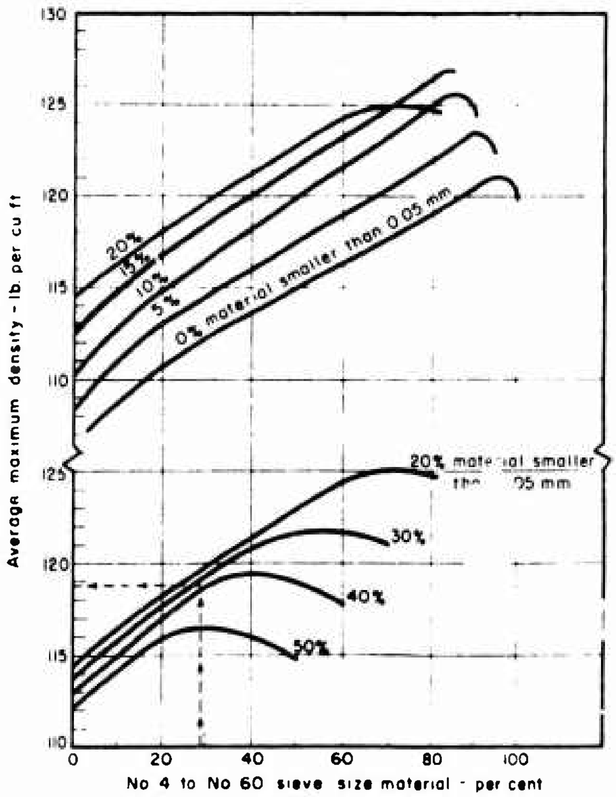


Fig. 35. Average maximum densities of soil-cement mixtures not containing material retained on the No. 4 sieve.

Step-by-Step Procedures

Short-cut test procedures involve:

1. Running a moisture-density test on a mixture of the soil and portland cement.
2. Determining the indicated portland cement requirement by the use of charts.
3. Verifying the indicated cement requirement by compressive-strength tests

Preliminary Steps

Before applying the short-cut test procedures, it is nec-

*Since this material has been taken directly from the Portland Cement Association text, figure numbers and certain other references in this Appendix will not be in agreement with other portions of this report.

essary (1) to determine the gradation of the soil, and (2) to determine the bulk specific gravity of the material retained on the No. 4 sieve.* If all the soil passes the No. 4 sieve, Method A should be used. If material is retained on the No. 4 sieve, Method B is used.

Method A

Step 1: Determine by test the maximum density and optimum moisture content for a mixture of the soil and portland cement.**

Note 1: Use Fig. 35 to obtain an estimated maximum density of the soil-cement mixture being tested. This estimated maximum density and the percentage of material smaller than 0.05 mm. (No. 270 sieve) can be used with Fig. 36 to determine the cement content by weight to use for the test.

Step 2: Use the maximum density obtained by test in Step 1 to determine from Fig. 36 the indicated cement requirement.

Step 3: Use the indicated cement factor obtained in Step 2 to mold compressive-strength test specimens† in triplicate at maximum density and optimum moisture content.

Step 4: Determine the average compressive strength of the specimens after 7 days' moist-curing.

Step 5: On Fig. 37, plot the average compressive-strength value obtained in Step 4. If this value plots above the curve, the indicated cement factor by weight, determined in Step 2, is adequate.

For field construction, use Figure 41 to convert this cement content by weight to a volume basis.

Note 2: If the average compressive-strength value plots below the curve of Fig. 37, the indicated cement factor obtained in Step 2 is probably too low. Additional tests will be needed to establish a cement requirement. These tests generally require the molding of two test specimens, one at the indicated cement factor obtained in Step 2 and one at a cement content two percentage points higher. The specimens are then tested by ASTM-AASHO freeze-thaw test procedures.

Method B

Step 1: Determine by test the maximum density and optimum moisture content for a mixture of the soil and portland cement.††

Note 3: Use Fig. 38 to determine an estimated maximum density of the soil-cement mixture being tested. This estimated maximum density, the percentage of material

*The short-cut tests do not apply to soils containing more than 50 per cent silt and clay smaller than 0.05 mm. and more than 20 per cent clay smaller than 0.005 mm., or to dark grey or black organic soils. These soils, as well as miscellaneous granular materials such as cinders, caliche, chat, chert, marl, red dog, scoria, shale, slag, etc., and soils containing material retained on the No. 4 sieve having a bulk specific gravity less than 2.45 should not be used but should be tested by the ASTM-AASHO procedures.

**Methods of Test for Moisture-Density Relations of Soil-Cement Mixtures, ASTM Designation D558-57; AASHO Designation T134-57.

†Specimens of either 2-in. diameter and 2-in. height or 4-in. diameter and 4.6-in. height may be molded. The 2-in. specimens shall be submerged in water for one hour before testing and the 4-in. specimens for four hours. The 4-in. specimens shall be capped before testing.

††Methods of Test for Moisture-Density Relations of Soil-Cement Mixtures, ASTM Designation D558-57; AASHO Designation T134-57.

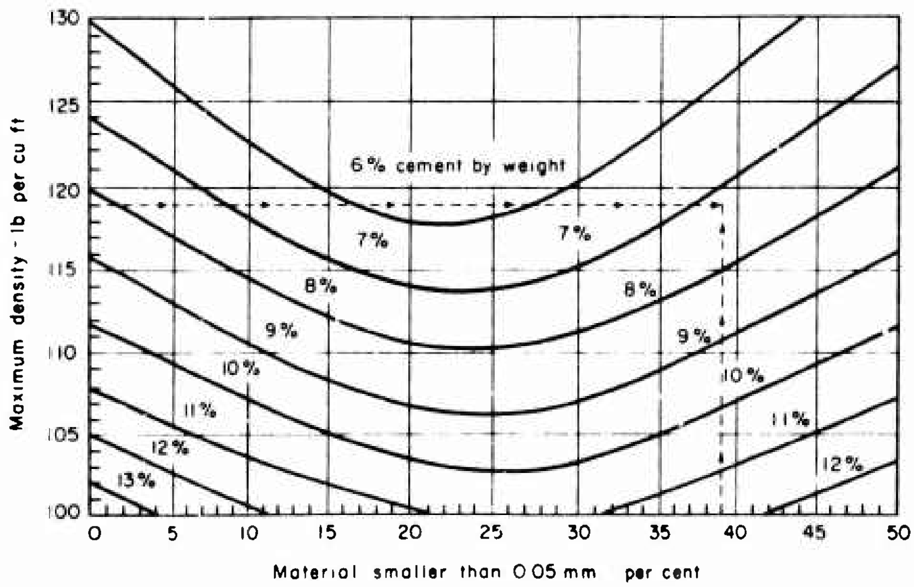


Fig. 36. Indicated cement contents of soil-cement mixtures not containing material retained on the No. 4 sieve.

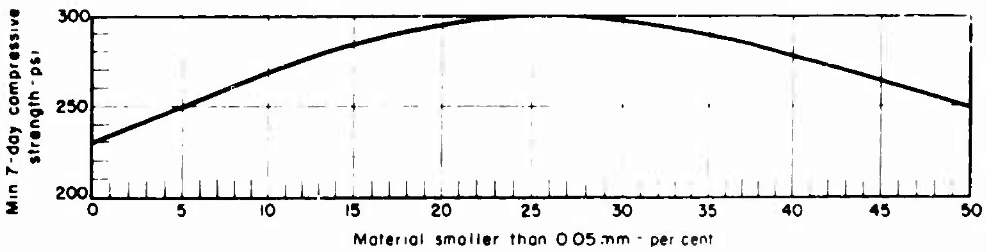


Fig. 37. Minimum 7-day compressive strengths required for soil-cement mixtures not containing material retained on the No. 4 sieve.

smaller than 0.05 mm. (No. 270 sieve), and the percentage of material retained on the No. 4 sieve can be used with Fig. 39 to determine the cement content by weight to use in the test.

The soil sample for the test shall contain the same percentage of material retained on the No. 4 sieve as the original soil sample contains, except that a maximum of 45 per cent is used. Also, ¾-in. material is the maximum size used. Should there be material larger than this in the original soil sample, it is replaced in the test sample with an equivalent weight of material passing the ¾-in. sieve and retained on the No. 4 sieve.

Step 2: Use the maximum density obtained by test in Step 1 to determine from Fig. 39 the indicated cement requirement.

Step 3: Use total material as described in Step 1 and the indicated cement factor obtained in Step 2 to mold compressive-strength test specimens* in triplicate at maximum density and optimum moisture content.

Step 4: Determine the average compressive strength of the specimens after 7 days' moist-curing.

Step 5: Determine from Fig. 40 the minimum allowable compressive strength for the soil-cement mixture. If the average compressive strength obtained in Step 4 equals or exceeds the minimum allowable strength, the indicated cement factor by weight obtained in Step 2 is adequate.

For field construction, use Figure 41 to convert this cement content by weight to a volume basis.

Note 4: If the average compressive-strength value is lower than the minimum allowable, the indicated cement factor obtained in Step 2 is probably too low. Additional tests as described in Note 2 are needed.

Example of Use of Short-Cut Test Procedures

Following is an example of the use of the short-cut procedures.

Preliminary tests determine the gradation of the soil and bulk specific gravity of the material, if any, retained on the No. 4 sieve. The data obtained from the tests are tabulated below. In this example, Method B should be used since the soil contains material retained on the No. 4 sieve.

Gradation:

Passing

No. 4 sieve	82 per cent
No. 10 sieve	77 per cent
No. 60 sieve	58 per cent
No. 200 sieve	37 per cent

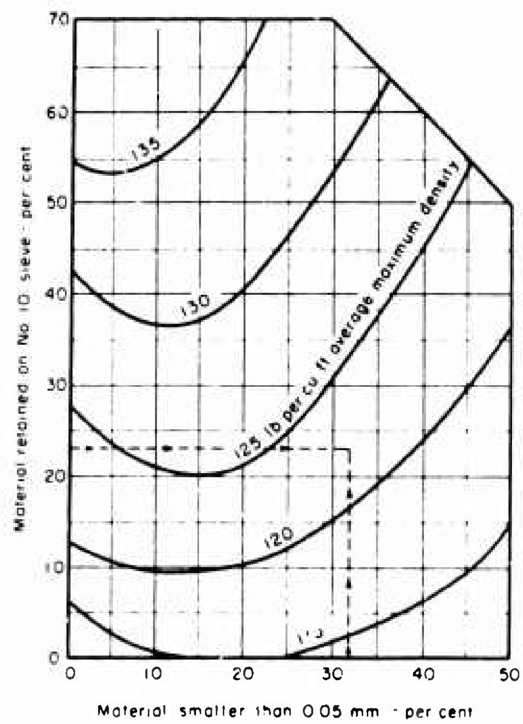


Fig. 38. Average maximum densities of soil-cement mixtures containing material retained on the No. 4 sieve.

*Specimens of 4-in. diameter and 4.6-in. height shall be molded. They shall be submerged in water for four hours and shall be capped before testing.

Smaller than
0.05 mm. (silt and clay combined) .... 32 per cent
0.005 mm. (clay) .... 13 per cent

Color: Brown

Bulk specific gravity of material retained on No. 4 sieve: 2.50.

Step 1: Fig. 38 indicates that the estimated maximum density of the soil-cement mixture is 122 lb. per cu.ft. since the soil contains 32 per cent material smaller than 0.05 mm. and 23 per cent material retained on the No. 10 sieve.

Fig. 39 is used to determine the cement content by weight to use in the moisture density test. Since the soil contains 32 per cent material smaller than 0.05 mm. and 18 per cent material retained on the No. 4 sieve, and since the estimated maximum density is 122 lb. per cu.ft., 6 per cent cement by weight is indicated.

Perform the moisture-density test.

For this example, assume the maximum density obtained by test to be 123.2 lb per cu.ft. at 10.2 per cent moisture.

Step 2: Fig. 39 indicates a cement factor of 6 per cent, using the calculated actual density of 123.2 lb. per cu.ft.

Step 3: Using total material and 6 per cent cement by weight, mold compressive-strength test specimens in triplicate at maximum density (123.2 lb. per cu.ft.) and optimum moisture (10.2 per cent).

Step 4: Determine the average 7-day compressive strength.

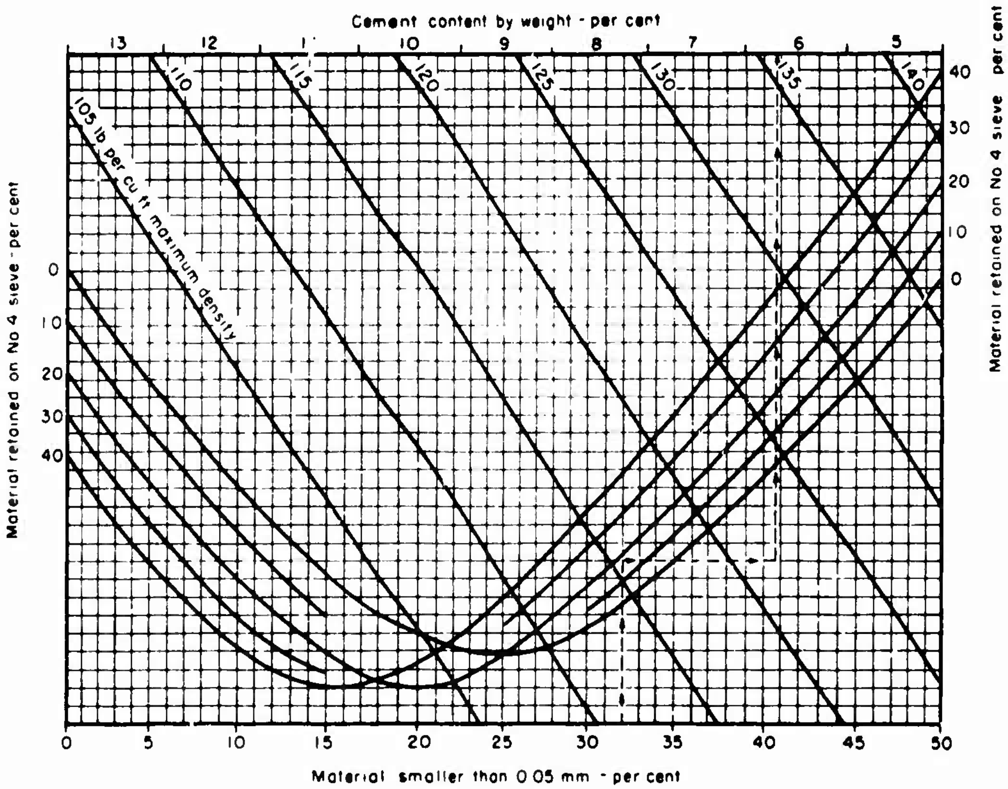
For this example, assume the average compressive strength to be 345 psi.

Step 5: Since the soil contains 32 per cent material smaller than 0.05 mm. and 18 per cent material retained on the No. 4 sieve, the minimum allowable compressive strength for this soil-cement mixture is 280 psi, as shown in Fig. 40. The average compressive strength of the mixture used in this example (345 psi), as obtained in Step 4, is higher than the minimum allowable strength. Therefore, the indicated cement content of 6 per cent by weight is adequate.

For field construction, Figure 41 shows that 6 per cent cement by weight is equivalent to 7.4 per cent cement by volume.

If the average compressive strength in Step 4 had been

Fig. 39. Indicated cement contents of soil-cement mixtures containing material retained on the No. 4 sieve.



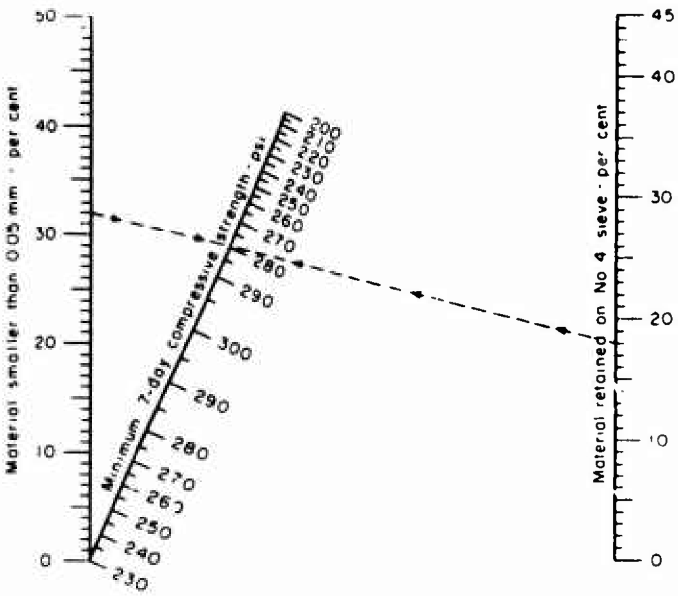


Fig. 40. Minimum 7-day compressive strengths required for soil-cement mixtures containing material retained on the No. 4 sieve.

lower than the minimum allowable strength, say 245 psi, 6 per cent cement by weight probably would not have been adequate. Additional testing would then have been required to establish the cement requirement for the soil. These tests would involve molding and testing freeze-thaw test specimens according to ASTM-AASHO procedures. Freeze-thaw specimens containing 6 and 8 per cent cement by weight would probably be adequate in this instance.

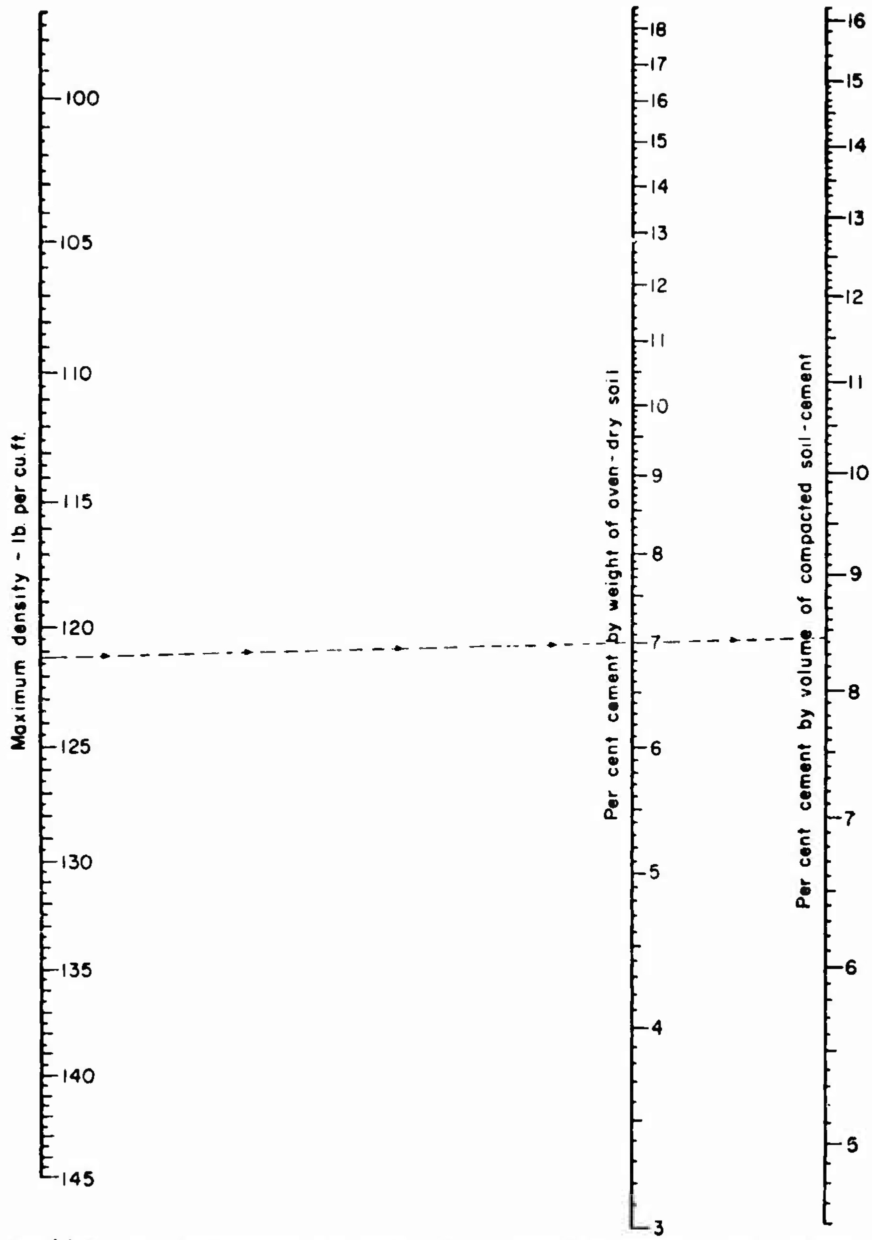


Fig. 41 Relation of cement content by weight of oven-dry soil to cement content by volume of compacted soil-cement mixture.

longed damp-mixing operations. However, the laboratory moisture-density test data for the soil-cement mixtures occurring on a project are sufficiently close to the field moisture-density test data that they can be used for estimating equipment needs and for setting up bid items in the contract proposal.

APPENDIX I

SELECTION OF CEMENT CONTENT FOR BASE COURSE SOIL-CEMENT MIXTURES

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THIS chapter will be of major interest to the laboratory engineer because it will assist him in determining what cement contents to investigate in the soil-cement tests. The field engineer and administrative engineer will also be interested because the properties of soil-cement mixtures and the relationships existing among these properties and various test values are discussed. Information is presented that will enable engineers to estimate probable cement factors so that job estimates can be made before any tests are made.

In order to obtain the maximum amount of information from the wet dry and freeze-thaw tests, it is important that the laboratory engineer design the soil-cement specimens properly. For instance, if specimens are designed with very high cement contents, they will all pass the wet-dry and freeze-thaw tests, and a minimum cement factor will not have been determined. On the other hand, if the specimens are designed with inadequate cement contents, they will all fail in the tests.

The principal requirement of a hardened soil-cement mixture is that it withstand exposure to the elements. Strength might also be considered a principal requirement; however, since most soil-cement mixtures that possess adequate resistance to the elements also possess adequate strength, this requirement is secondary.

Therefore, in a study to determine when a certain soil-cement mixture has been adequately hardened, the requirement of adequate resistance to exposure is the first considered. That is, will the hardened soil-cement mixture withstand the wetting and drying and the freezing and thawing cycles of nature and still maintain at least the stability inherent in the mass at the time the roadway was opened to traffic?

For instance, consider a hypothetical road subgrade made from a clay loam soil without cement, packed to maximum density at a moisture content slightly less than its optimum moisture content. This mass can withstand relatively heavy loads without failure, although it cannot offer much resistance to abrasive forces.

The same soil mixed with cement and compacted to maximum density at optimum moisture content will have stability before the cement hydrates at least equal to that of the raw soil.

But consider the two cases at a later date under a condition of slow drainage when moisture, by capillary action or in some other manner, has permeated the masses. The voids in the raw soil become filled with water and the soil loses the original inherent physical stability that was built into it by compaction to maximum density. This is not so, however, with the adequately hardened soil-cement mixture, which has continually increased in stability since its construction because of cement hydration and resultant cementation. Its air voids will become filled with water too, but its stability will still be much greater than that built into it originally.

The next important requirement to consider is economy. Available data indicate that about 85 per cent of all soils likely to be used for soil-cement can be adequately hardened by the addition of 14 per cent cement or less. To determine whether or not a soil falls into this category would not require much testing. However, more than 50 per cent of all soils so far tested for soil-cement require only 10 per cent cement or less for adequate hardening. To identify these soils requires more testing. Since soil-cement is in the low-cost paving field, the testing engineer on large jobs should determine by test the minimum quantity of cement that can be safely used with each soil. By this procedure the lowest-cost soil-cement construction possible will be obtained.

Estimating Cement Requirements

The following information will aid the engineer in estimating cement requirements of the soils proposed for use and in determining what cement factors to investigate in the laboratory tests.

As a general rule, it will be found that the cement requirement of soils increases as the silt and clay content increases, gravelly and sandy soils requiring less cement for adequate hardness than silt and clay soils.

The one exception to this rule is that poorly graded, one-size sand materials that are devoid of silt and clay require more cement than do sandy soils containing some silt and clay.

In general, a well-graded mixture of stone fragments or gravel, coarse sand, and fine sand either with or without small amounts of feebly plastic silt and clay material will

*Since this material has been taken directly from the Portland Cement Association text, figure numbers and certain other references in this Appendix will not be in agreement with other portions of this report.

require 5 per cent or less cement by weight. Poorly graded one-size sand materials with a very small amount of non-plastic silt, typical of beach sand or desert blow sand, will require about 9 per cent cement by weight. The remaining sandy soils will generally require about 7 per cent. The nonplastic or moderately plastic silty soils generally require about 10 per cent cement by weight, and plastic clay soils require about 13 per cent or more.

Table 1 gives the usual range in cement requirements for subsurface soils of the various AASHO* soil groups. "A" horizon soils may contain organic or other material detrimental to cement reaction and may require higher cement factors. For most A horizon soils the cement content in Table 1 should be increased four percentage points if the soil is dark grey to grey and six percentage points if the soil is black. It is usually not necessary to increase the cement factor for a brown or red A horizon soil. Testing of "poorly reacting" sandy surface soils is discussed in detail in Chapter 8. These cement contents can be used as preliminary estimates, which are then verified or modified as additional test data become available.

TABLE 1. Cement Requirements of AASHO Soil Groups

AASHO soil group	Usual range in cement requirement, per cent by vol.	Usual range in cement requirement, per cent by wt.	Estimated cement content and that used in moisture-density test, per cent by wt.	Cement contents for wet-dry and freeze-thaw tests, per cent by wt.
A-1-a	5–7	3–5	5	3-5-7
A-1-b	7–9	5–8	6	4-6-8
A-2	7–10	5–9	7	5-7-9
A-3	8–12	7–11	9	7-9-11
A-4	8–12	7–12	10	8-10-12
A-5	8–12	8–13	10	8-10-12
A-6	10–14	9–15	12	10-12-14
A-7	10–14	10–16	13	11-13-15

Step-by-Step Procedure

The following procedure will prove helpful to the testing engineer in setting up cement contents to be investigated:

Step 1: Determine from Table 1 the preliminary estimated cement content by weight based on the AASHO soil group.

Step 2: Use the preliminary estimated cement content obtained in Step 1 to perform the moisture-density test.

Step 3: Verify the preliminary estimated cement content by referring to Table 2 if the soil is sandy or to Table 3 if it is silty or clayey. These tables take into consideration the maximum density and other properties of the soil, which permits a more accurate estimate. In the case of A horizon soils, the indicated cement factor should be increased as discussed above for Table 1.

Sandy soils:

(1) Using the percentage of material smaller than 0.05 mm., the percentage of material retained on the No. 4 sieve, and the maximum density obtained by test in Step 2, determine from Table 2 the estimated cement content.

(2) Mold wet-dry and freeze-thaw test specimens at the estimated cement content by weight obtained in (1) and at cement contents two percentage points above and below that cement factor.

Silty and clayey soils:

(1) Using the percentage of material between

*Charts and tables for use in classifying soils by the American Association of State Highway Officials Soil Classification System (AASHO Designation: M 145-49) are given in the appendix.

TABLE 2. Average Cement Requirements of B and C Horizon Sandy Soils

Material retained on No. 4 sieve, per cent	Material smaller than 0.05 mm., per cent	Cement content, per cent by wt. — Maximum density, lb. per cu.ft.: 105–109	110–114	115–119	120–124	125–129	130 or more
0–14	0–19	10	9	8	7	6	5
	20–39	9	8	7	7	5	5
	40–50	11	10	9	8	6	5
15–29	0–19	10	9	8	6	5	5
	20–39	9	8	7	6	6	5
	40–50	12	10	9	8	7	6
30–45	0–19	10	8	7	6	5	5
	20–39	11	9	8	7	6	5
	40–50	12	11	10	9	8	6

TABLE 3. Average Cement Requirements of B and C Horizon Silty and Clayey Soils

AASHO group index	Material between 0.05 mm and 0.005 mm, per cent	Cement content, per cent by wt.						
		Maximum density, lb. per cu.ft.						
		90–94	95–99	100–104	105–109	110–114	115–119	120 or more
	0–19	12	11	10	8	8	7	7
	20–39	12	11	10	9	8	8	7
0–3	40–59	13	12	11	9	9	8	8
	60 or more	—	—	—	—	—	—	—
	0–19	13	12	11	9	8	7	7
	20–39	13	12	11	10	9	8	8
4–7	40–59	14	13	12	10	10	9	8
	60 or more	15	14	12	11	10	9	9
	0–19	14	13	11	10	9	8	8
	20–39	15	14	11	10	9	9	9
8–11	40–59	16	14	12	11	10	10	9
	60 or more	17	15	13	11	10	10	10
	0–19	15	14	13	12	11	9	9
	20–39	16	15	13	12	11	10	10
12–15	40–59	17	16	14	12	12	11	10
	60 or more	18	16	14	13	12	11	11
	0–19	17	16	14	13	12	11	10
	20–39	18	17	15	14	13	11	11
16–20	40–59	19	18	15	14	14	12	12
	60 or more	20	19	16	15	14	13	12

0.05 mm and 0.005 mm, the AASHO group index, and the maximum density obtained by test in Step 2, determine from Table 3 the estimated cement content.

(2) Mold wet-dry and freeze-thaw test specimens at the estimated cement content obtained in (1) and at cement contents two percentage points above and below that cement factor.

To help in determining how well the soil reacts, it is advantageous to save half of the last moisture-density test specimen and to place it in an atmosphere of high humidity for inspection daily. This half specimen, called the

Fig. 5. Soil-cement specimens saved from tail end of moisture-density test procedure. Rate of hardening of the soil-cement mixture is investigated from day to day with a dull-pointed instrument.

"tail-end" specimen (see Fig. 5), is obtained during the usual procedure of cutting the last specimen of the moisture-density test in half vertically (details are given on page 20) so that a representative moisture sample can be taken. The criteria used in the rapid test procedure, as discussed in Chapter 7, can be used to judge the hardness of the tail-end specimen. Generally, tail-end specimens are satisfactorily hardened in two to four days and it is not uncommon for them to be satisfactory a day after molding.

A study of compressive-strength data, as discussed in Chapter 4, is also helpful in checking the estimated cement factor.

Miscellaneous Soils

A number of miscellaneous materials or special types of soils, such as caliche, chert, cinders, scoria, shale, etc., have been used successfully in soil-cement construction. In some cases these materials have been found in the roadway or street that was to be paved with soil-cement; in other cases, in order to reduce the cost of the project, they have been used as borrow materials to replace soils that required high cement contents for adequate hardening.

The procedure for testing miscellaneous materials is the same as that used for regular soils. Average cement requirements of a number of miscellaneous materials and cement contents to be investigated in the laboratory tests are given in Table 4. As test data are accumulated and experience is gained with local miscellaneous materials, it may be found that future testing can be reduced or eliminated for similar materials.

TABLE 4. Average Cement Requirements of Miscellaneous Materials

Type of miscellaneous material	Estimated cement content and that used in moisture-density test		Cement contents for wet-dry and freeze-thaw tests, per cent by wt.
	per cent by vol.	per cent by wt.	
Shell soils	8	7	5- 7- 9
Limestone screenings	7	5	3- 5- 7
Red dog	9	8	6- 8-10
Shale or disintegrated shale	11	10	8-10-12
Caliche	8	7	5- 7- 9
Cinders	8	8	6- 8-10
Chert	9	8	6- 8-10
Chat	8	7	5- 7- 9
Marl	11	11	9-11-13
Scoria containing material retained on the No. 4 sieve	12	11	9-11-13
Scoria not containing material retained on the No. 4 sieve	8	7	5- 7- 9
Air-cooled slag	9	7	5- 7- 9
Water-cooled slag	10	12	10-12-14

APPENDIX J

pH TEST TO DETERMINE LIME REQUIREMENTS FOR LIME STABILIZATION

Materials:

1. Lime to be used for soil stabilization

Apparatus:

1. pH meter (the pH meter must be equipped with an electrode having a pH range of 14)
2. 150 ml. (or larger) plastic bottles with screw-top lids
3. 50 ml. plastic beakers
4. CO_2 - free distilled water
5. Balance
6. Oven
7. Moisture cans

Procedure:

1. Standardize the pH meter with a buffer solution having a pH of 12.45.
2. Weigh to the nearest 0.01 gms. representative samples of air-dried soil, passing the No. 40 sieve and equal to 20.0 gms. of oven-dried soil.
3. Pour the soil samples into 150 ml. plastic bottles with screw-top lids.
4. Add varying percentages of lime, weighed to the nearest 0.01 gm., to the soils. (Lime percentages of 0, 2, 3, 4, 5, 6, 8 and 10, based on the dry soil weight, may be used.)
5. Thoroughly mix soil and dry lime.

6. Add 100 ml. of CO_2 - free distilled water to the soil-lime mixtures.
7. Shake the soil-lime and water for a minimum of 30 seconds or until there is no evidence of dry material on the bottom of the bottle.
8. Shake the bottles for 30 seconds every 10 minutes.
9. After one hour, transfer part of the slurry to a plastic beaker and measure the pH.
10. Record the pH for each of the soil-lime mixtures. The lowest percent of lime giving a pH of 12.40 is the percent required to stabilize the soil. If the pH does not reach 12.40, the minimum lime content giving the highest pH is that required to stabilize the soil.

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